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**ENGINEERING
PROPERTIES OF SOIL**

ENGINEERING PROPERTIES OF SOIL

BY

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PREFACE

Soil is the oldest and probably the most used of engineering materials. The extent to which engineering works render the service contemplated in their design depends largely on the properties of the soil on, in, or of which they are constructed. Yet working knowledge of the structural properties of soil and the methods for their evaluation has been meager compared with knowledge of the properties and uses of newer materials such as concrete, steel, and brick. Discussion of soil performance in undergraduate engineering courses has been confined, generally, to theories regarding the pressures to be provided for in the design of retaining walls and abutments—and the safe loads permitted by building codes of the different municipalities in the design of footings to support buildings on foundation soils of the various types.

The decade just past saw rapid progress in the understanding of physical phenomena responsible for the properties, such as shrinkage, expansion, and capillarity, which control the structural stability of soil. This stimulated a desire to apply the new knowledge of soils in the design and construction of foundations and structures.

The increased interest of the engineering profession in soils investigations resulted in the development of many new ideas, theories, and methods for determining the properties of soil. In many cases the espoused theories regarding the performance of soil were controversial, and their practical application required the use of higher mathematics generally included in postgraduate rather than undergraduate courses.

Both the practicing engineer and the instructor were therefore compelled to assume an attitude of hopeful patience regarding the outcome of the investigators' efforts and, in general, to await the simplification of procedures for applying the results furnished by soil studies in the design and construction of engineering work. As a result, courses of instruction in the new and rapidly developing subject could not well be included in the regular college and university curriculums.

The results of research on road soils becoming available caused the subject of soil investigations to assume more and more a practical instead of an experimental status. Coincidentally, there came requests from interested engineers, teachers, and others for instruction in those fundamentals the validity of which had been demonstrated in practical construction. Accordingly, arrangements were made by the Bureau of Public Roads to provide the desired instruction at the subgrade-soil testing laboratory of the bureau, located at Arlington, Va.

Material presented in published reports by members of the bureau's staff and cooperating agencies was used in these courses of instruction. Gradually this material has assumed a definite and more or less stable form, so that it can now be embodied in a textbook.

That our knowledge of soils for engineering purposes has crystallized into definite practical form is evidenced by the present status of soil studies. Tests are now available for determining the relations between the moisture content of a soil and the maximum density obtainable by a selected method of compaction; the effect of admixtures and methods of compaction for increasing or decreasing the maximum obtainable density; the resulting consistency of the mixture in both confined and unconfined state; and the extent to which the acquired density of the compacted soil will be retained under varying conditions of load and moisture. Substantial progress has been made in the design of soil mixtures, the use of bituminous-surface treatments, the stabilization of the moisture content by means of treatment with deliquescent chemicals, and the construction of stabilized embankments for use as earth dams, fills, and levees.

The validity of the various tests for disclosing characteristic soil properties has been adequately established by observations of the performance of structures in service. Mathematical relationships have been found to exist between the results furnished by the so-called simplified or indicator tests and those furnished by the more elaborate compression tests used in the determination of the properties of soils to be used as foundations for bridges, fills, etc. Procedures for surveying, mapping, and testing subgrade soils were adopted as standard methods by the American Association of State Highway Officials in 1934 and as tentative standards by the American Society for Testing Materials in 1935.

The subject matter of this book is presented as an application of the principles of physics, chemistry, and mechanics included in the undergraduate courses of engineering. In the demonstrations of the various theories and physical laws, the more elaborate mathematical relations of postgraduate status have been eliminated in favor of simpler arithmetical and algebraic expressions.

Publications by the soil scientist, the soil physicist, the agronomist, the pedologist, and the geologist have been drawn upon extensively.

Of these publications the following should be continuously available to the student for reference: "Soils—Their Origin, Constitution and Classification," by G. W. Robinson; "Soil Conditions and Plant Growth," by Edward J. Russell; "Textbook of Geology," by Louis V. Pirsson and Charles Schuchert; "The Chemistry and Physics of Clays and Other Ceramic Materials," by Alfred B. Searle; "Applied Colloid Chemistry," by Wilder D. Bancroft; "Colloid and Capillary Chemistry," by Herbert Freundlich; and "Soils," by E. W. Hilgard.

General information on the properties of soil in relation to the design and construction of engineering work and general descriptions and the significances of the methods used in soil examination and the utilization of test results in the design of stable, durable, and economic structures are included. This material is arranged for the use of instructors in engineering materials, engineering students, and practicing engineers who desire a very general but complete conception of the physical characteristics of soils and their influence on the performance of soil as an engineering material, and also the engineer and road builder who require an intimate and comprehensive knowledge of the engineering properties of soil and tests for disclosing them.

Numerical examples are included to demonstrate mathematical relationships. There are included in appendices information to supplement the text material, as follows:

Appendix I. Conversion tables of United States and metric systems of measurement.

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Appendix IV. Glossary of terms used by the U. S. Bureau of Chemistry and Soils in soil surveys.

Included also is a complete bibliography of reports to which reference is made in the text by means of figures in parentheses.

The authors desire to express appreciation to Mr. L. A. Palmer for assistance in the treatment of the mathematical theories of pressure distribution in soils and soil consolidation; to Mr. Paul Rapp for assistance in the presentation of the principles of colloid chemistry; and to Messrs. Herbert Fairbank, E. F. Kelley, Roy W. Crum, and Dean John R. Lapham for review of the manuscript.

C. A. HOGENTGLER.

WASHINGTON, D. C.,
January, 1937.

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ENGINEERING PROPERTIES OF SOIL

INTRODUCTION

Present knowledge of soil represents the composite results of efforts made independently in different fields of endeavor to determine the manner in which this material originated and the properties that control its performance in agriculture, industry, and engineering.

Studies of the physical character, structural properties, and mechanical grading of soil are of particular interest to the engineer. Contributions of this kind have been furnished principally by the geologist, the agronomist, the industrial chemist, the highway engineer, and the structural engineer.

The geologist, concerned with study of soils as part of the earth as a whole, has contributed much of the data on the formation of individual soil materials from the parent rock and the methods whereby these materials were mixed and deposited in the layers of the earth's surface.

The agronomist, interested in the properties of soils influencing plant growth and the methods by which the natural soil materials may be improved from the agricultural standpoint, has furnished information on the texture, water-holding properties, organisms, and chemical constituents and on the improvement of the natural soil by drainage, irrigation, or treatment with fertilizers, to increase the yield and quality of plants.

The industrial chemist's contributions result from his efforts to utilize soil materials in the manufacture of Portland cement, glass, ceramic articles, and other products of commerce.

Highway and structural engineers have studied soil as an aggregate for cement concrete, bituminous mixtures, and stabilized soil mixtures; as a foundation for road surfaces, buildings, bridges, fills, etc.; and as a construction material for use in earth dams, levees, and embankments.

Physical Characteristics Studied First.—Soil mechanics may be defined as that branch of science which deals with the performance of soil as a structural material. Included among the physical principles upon which it is based are those which control the performance of fluids at rest, fluids in motion, and the deformations of both elastic and plastic bodies due to stress. It can be said that its origin dates back as far as the year 250 B.C., at which time Archimedes is credited by Merriman (1) with having established some of the principles of hydraulics.

Among other hydraulicians who contributed basic theories are Galileo, who studied the flotation of bodies in water, 1630; Pascal, who studied the influence of atmospheric pressure on the rise of liquids in a vacuum, 1650; Mariotte, who was the first to consider the influence of friction on the flow of liquids in containers, 1680; and Newton, who, in 1685, was the first to note the contraction of a jet issuing from an orifice (1).

More directly applicable to our present conception of the subject are the following: Hooke's law of stress and strain, 1676; theories of the stability of retaining walls advanced by Coulomb in 1773, by Navier in 1826, and by Rankine in 1856; Mohr's theory of rupture, 1868; Darcy's law of the flow of water through soils, 1856; and Stokes's law of the velocity of solids falling through liquids, 1856. Although these theories had been available for a much longer time, substantial contributions to our present conception of soil technology date from the very recent time when the importance of the effect of the surface tension of contained moisture and of the physical character of the soil was first recognized.

In the year 1892, Milton Whitney, then chief of the U. S. Weather Bureau, concluded that physical characteristics serve better than chemical analyses for indicating the performance of soils (2). At the same time this pioneer explained in detail the effect of surface tension for contracting, expanding, and supplying cohesion in soils; suggested a formula expressing the relation between surface tension, diameter of capillaries, and height of capillary rise; and described a method for determining the moisture content of natural soils *in situ* by means of the changing electrical resistance between two installed metal-plate terminals.

In 1898, Dr. George E. Ladd (3, 4) advanced two other basic conceptions: (a) that silt consists of bulky grains and clay of

scalelike particles and (b) that shrinkage of soil is caused by surface tension when water recedes in the soil capillaries.

Tests for indicating the capillary lift of soils were described as early as 1896 (5). The moisture-equivalent test for disclosing the water-holding properties of soils was used by the U. S. Bureau of Soils as early as 1907 (6).

In 1911 A. Atterberg (7, 8), a Swedish scientist, suggested simple test procedures for disclosing the plastic and the shrinkage properties of soils. A. C. Rose (9, 10) of the U. S. Bureau of Public Roads, in 1924, first correlated road-surface performance with the results of tests devised to disclose the physical properties of subgrade soils.

In 1913, the Board of Directors of the American Society of Civil Engineers (11) appointed a soils committee whose work was to determine and report on (a) present practice in the bearing value of soils and (b) physical characteristics of soils in relation to engineering structures.

A committee on the subgrade and its relation to road surfacing and traffic, with Gen. Coleman Dupont as chairman and Charles M. Upham, Henry Shirley, and F. H. Eno included in the membership, was appointed by the Federal Highway Council, 1920. This committee was later merged into the Committee on Structural Design of the Highway Research Board.

The Ohio Good Roads Federation just before this time appointed D. S. Humphrey, Will P. Blair, and W. A. Alsdorf as members of a committee to inaugurate and direct a study of subgrades (12).

Thomas H. MacDonald, Chief, inaugurated research on subgrade soil at the U. S. Bureau of Public Roads in 1920, with A. T. Goldbeck and Ira B. Mullis in charge. In connection with the Bates road tests, Clifford Older and H. F. Clemmer (13) investigated the bearing power and drainage of soils. Charles M. Upham and H. F. Janda (14) in North Carolina studied the characteristics of capillary moisture in soils.

Daniel E. Moran (15) reports making consolidation tests on undisturbed samples of clay and other foundation materials as early as 1910. Subsequently with the publication of Terzaghi's treatise on soil mechanics, *Erdbaumechanik*, in 1925 (16), mathematical formulas were made available for determining quantitatively the effects of surface tension on the performance of

soils. Terzaghi's theory of consolidation opened up the way to determining properties such as compressibility, expansibility, and permeability. With such information, furnished by tests on small samples in the laboratory, it became possible to estimate the ultimate settlements to be expected in foundations constructed on soft undersoils and the rate at which they would occur.

Investigations in the field of soil mechanics have been made also at the Iowa State College by John Griffith (17), at the University of Iowa by C. C. Williams, at the University of California by Raymond E. Davis (18), at the University of Michigan by W. S. Housel (19), and at Yale University by D. P. Krynine (20). Included in basic soil studies are also those on drainage by W. J. Schlick, Iowa State College (21); the occurrence and movement of ground waters by O. E. Meinzer, U. S. Geological Survey (22); and the hydraulic method of dam construction by Harry Hatch (23) of the Water Works Department of Springfield, Mass.

Grading of Road Soils Studied.—Investigations begun in 1906 by Dr. C. M. Strahan (24), then county engineer of Clark County, Georgia, mark the first consistent effort to correlate the performance of topsoils in road surfaces with their gradings as determined by mechanical analyses.

The name "topsoil" was applied to soil skimmed from the surface of fields to shallow depths. It was thought that from long cultivation such deposits had undergone weathering, water separation, and the probable influence of organic acids which gave them their superior consistency and binding stability.

In Dr. Strahan's work samples were obtained from short stretches of existing roads where firmness and water-resisting qualities were noted in contrast with ordinary conditions of dirt roads. Fifty or more counties were visited during the study. The data from laboratory tests on the samples collected were correlated with the road behavior under varying weather conditions and traffic, and the conclusions reached became the basis for the selection of materials and the adoption of laboratory standards.

The first statement of the probable mode of action of road soils under varying conditions of traffic and weather was prepared by Dr. Strahan in 1914 for distribution at the Fourth American Road Congress held that year in Atlanta, Ga.

After the passage of the Federal Aid Act in 1916, the first conference of state highway-testing engineers and chemists was called to meet in Washington, February, 1917, under the auspices of the Bureau of Public Roads, to consult concerning standards and specifications for the various road-building materials.

A committee consisting of C. B. Scott, Virginia; I. B. Mullis, North Carolina; and C. M. Strahan, Georgia, on semigravel, topsoil, and sand-clay aggregates recommended gradings for the three classes of topsoils: hard or class *A*, medium or class *B*, and soft or class *C*.

These gradings remained unchanged in subsequent reports of the work by Dr. Strahan, the last of which was published in September, 1929 (25).

Cooperative Efforts Inaugurated.—In 1926 the Bureau of Public Roads, through its Division of Tests headed by E. F. Kelley, initiated an attack on the road-soil problem in which the efforts of the highway engineer were supplemented by the cooperation of the soil physicist, the pedologist, and the geologist. The senior author was placed in charge of this work, with the other authors assisting. More or less informal cooperation was maintained with other federal bureaus and the various state highway departments. Cooperation of a more formal character was established with the various colleges and universities engaged in the study of specific problems. Cooperation in the work of the various national professional and engineering societies was maintained by membership of the bureau's staff on committees engaged in the study of soils.

Among the cooperative studies with other federal departments were investigations of the properties of foundation soils with the Bureau of Yards and Docks, U. S. Navy Department; development of methods of test and control for use in the construction of earth dams and similar embankments, with the Forest Service; and research on methods of making soil surveys for highway purposes, in which W. I. Watkins, soil surveyor of the Bureau of Chemistry and Soils, cooperated.

Among the special researches of universities, participated in by the bureau, were studies of frost heave by Stephen Taber, at the University of South Carolina; of soil mechanics by Charles Terzaghi, Glennon Gilboy, and Arthur Casagrande at the

Massachusetts Institute of Technology; and of soil stabilization by F. H. Eno at the Ohio State University.

Road-soil Classification and Theory of Stability Developed.—

Among the results of the combined efforts expended in the research on soils for engineering purposes, a report (26) was published in 1929 which added two new developments on the subject of soil analysis:

1. A procedure for testing soils in the laboratory and classifying them in groups according to a fixed classification schedule. All the soils in a given group were shown to have similar characteristics with regard to drainage and to be similar in their effect on road design.

2. A mathematical theory of stability which gave some conception of the relative influence of such factors as the granular fraction, the cohesive fraction, the moisture content of the soil, and the weight- and the load-distributing properties of the road surface.

This grouping was developed in response to the general question, What conspicuous performances of soil are of interest in the design, the construction, and the maintenance of highways? Answers to this question formed the basis of the group designations. Thus the soil grouping proposed had the unique feature of indicating performance under existing conditions of traffic, construction, and climate. In this it differed from the older classifications of the geologist based upon geological origin and that of the agronomist based upon the pedologic development.

The ultimate aim of this classification is to have each group designation signify characteristic performance of soils, the methods of improving performance, and the corresponding design and construction requirements. In this connection the gradings suggested by Dr. Strahan, modified in the light of new experience and supplemented by the results of simple physical tests, were adopted for use in the identification of the quality of soil mixtures and to disclose the effect of admixtures for improving the quality of the poorer soils. This is described in a series of reports published in 1931, later reprinted by the bureau under one cover with the title "Reports on Subgrade Soil Studies." This publication represented the closest approach to a textbook then existing on the subject, which by this time had developed into proportions having all the semblance of a new branch of science (27).

Chemicals Used First to Suppress Dust.—Along with the development of the use of bituminous materials for the suppres-

sion of dust the use of chemicals was also suggested. As early as 1907, Austin Thomas Byrne (28) discussed the use of sea water and deliquescent salts, and in 1909 the Bureau of Public Roads investigated the use of waste sulphite liquor and molasses (29). Calcium chloride came into use as a dust layer on macadam streets in about 1912. The value of this substance as a dust layer for gravel roads was quite generally recognized by 1916.

The possibilities of sodium chloride as a dust layer were suggested by R. H. Phillips in 1919 (30). It was used in experiments on the Wendover cutoff in 1924 (31) and in Nova Scotia in 1931 (32).

A report of an investigation of calcium chloride as a dust palliative by the Highway Research Board (33) first suggested the possibilities of this chemical for stabilizing the moisture content of graded materials in low-type roads. This investigation included observations and tests on experimental roads in South Carolina, Missouri, and Nebraska. Supplemental tests were made to obtain quantitative data on the rate of evaporation from treated and untreated soil samples, the effect of rewetting the surface on the rate of evaporation, and the leaching of calcium chloride by moisture.

A report by W. R. Collings and L. C. Stewart (34) gives the results of traffic tests on test roads. This investigation included tests of various combinations of soils in road sections constructed on a large indoor track on an earth subgrade.

Encouraging results furnished by these large-scale experiments were followed by the construction of stabilized-soil road surfaces in a number of states, with the largest mileages in Michigan; Indiana; and Onondaga County, New York.

Chemical Admixtures Used as Stabilizers.—The stabilization of soils by means of admixtures other than soil materials was tried in 1924 and 1925 (35) by Prof. F. H. Eno, who used both hydrated lime and Portland cement as admixtures to improve subgrade soils in experimental road sections in Ohio. In 1929 Professor Eno used salt, hydrated lime, calcium chloride, sodium silicate, kerosene, coal tar, crude oil, crankcase oil, stone dust, and granulated slag in experimental sections of subgrades of six traffic-bound roads in Ohio. The treatments were carried to a depth of 3 in. in the subgrade (36).

PRESENT STATUS OF SUBGRADE

COMPILED

SUB COMMITTEE ON SUBGRADES,
AMERICAN ASSOCIATION OF

State	Laboratory Tests						Field Survey				Use of Data		Data Reported	
	Mechanical Analysis	Plasticity Tests	Centrifuge Moisture Equilibrant Field	Moisture Equilibrant Field	Shrinkage Tests	Selection of Test Samples	Soil Profile Layer	Designation Soil Profile	Type and Series	Inspection by Resident Engineer	Corrective Treatment for subgrades; select 10% of fill material	Selection of soil materials for base and road	On Preliminary Plans	To Design Engineer
														All Projects
														Part of Projects
Alabama	○					○				○				○
Arizona	⊗	⊗	⊗	⊗	⊗	⊗				○				○
Arkansas														
California	○	○		○	○	○	○	○		○				○
Colorado														
Connecticut	○	○	○	○	○					⊗				○
Delaware														
District of Columbia	⊗	⊗	⊗	⊗	⊗		⊗			⊗				⊗
Florida														
Georgia	⊗					○				⊗				○
Iowa														
Illinois	○	○				○	○	○		⊗				○
Indiana	○	○				○								
Iowa	⊗	⊗	⊗	⊗	⊗	⊗	○	○		○				○
Kansas	⊗	⊗	⊗	⊗	⊗	⊗	○	○		○				○
Kentucky														
Louisiana	○	○				○				⊗				○
Maine														
Maryland	○	○		○	○	○	○			⊗				○
Massachusetts										⊗				
Michigan		○			○	○		⊗		⊗				⊗
Minnesota	⊗	⊗	⊗	⊗	⊗	○	⊗	○		○				○
Mississippi	⊗	⊗	⊗	⊗	⊗	○				○				○
Missouri	⊗	⊗	+	⊗	⊗		⊗			⊗				⊗
Montana														
Nebraska	○	○		○	○	⊗				⊗				
Nevada														
New Hampshire	⊗	⊗	⊗	⊗	⊗		⊗			⊗				⊗
New Jersey														
New Mexico	○			⊗	⊗	⊗				⊗				
New York														
North Carolina	○	○		○	○	○				⊗				○
North Dakota														
Ohio	⊗	⊗	⊗	⊗	⊗	○	+			⊗				○
Oklahoma		○	○	○	○	○				⊗				○
Oregon														
Pennsylvania	⊗	⊗	⊗	⊗	⊗		⊗			⊗				
Rhode Island														
South Carolina	⊗					⊗				○				○
South Dakota														
Tennessee	⊗	⊗	⊗	⊗	⊗		⊗			○				○
Texas	⊗	⊗	⊗	⊗	⊗	○	+			○				○
Utah														
Vermont						⊗								○
Virginia														
Washington	○			○	○	○				○				○
West Virginia	⊗	⊗	⊗	⊗	⊗	○	⊗			○				⊗
Wisconsin	⊗	⊗	⊗	⊗	⊗		⊗			⊗				○
Wyoming				○	○									
Total	19	17	16	19	19	10	11	1	12	23	6	5	2	5
Total	31	28	17	28	29	27	13	5	24	34	25	17	5	21

CHART

INVESTIGATIONS IN HIGHWAY DEPARTMENTS

BY
COMMITTEE ON MATERIALS
STATE HIGHWAY OFFICIALS

Principal value of subgrade investigations and special problems requiring investigation and treatment.	State
Subgrade treatment; sand-clay, top soil, clay-gravel, chert for base courses.	Alabama
Oiled roads.	Arizona
Base courses for low cost roads. Fill subsidence.	Arkansas
Shrinkage and low support. Frost heils. Fill erosion.	California
Settlement and volume change in fills.	Colorado
Drainage and landslides.	Connecticut
Subgrade treatment where clay content > 30% Sand for bituminous road mix.	Delaware
Low cost roads. Excavation and backfill at soft spots.	District of Columbia
Subbases and drainage.	Florida
Low cost roads. Compaction of subgrade.	Georgia
Stabilized roads (Calcium chloride, rock salt, asphalt emulsion).	Idaho
Subgrade treatment for gravel and oil gravel roads. Frost heils.	Illinois
Warping of concrete pavements.	Indiana
Base courses for low cost roads. Fill subsidence.	Iowa
Subbases employed at weak spots.	Kansas
Low cost roads. Frost heils.	Kentucky
Shrinkage.	Louisiana
Frost heaving. Fill settlement.	Maine
Frost heaving. Crack control stabilized roads. Base courses for low cost roads.	Maryland
Selection of subgrade material. Base courses.	Massachusetts
Low cost roads. Fill construction.	Michigan
Corrective treatment; subbases.	Minnesota
Fill construction. Bituminous mat central. Raising grade, compaction drainage.	Mississippi
Field moisture equivalent and shrinkage tests used as criteria.	Missouri
Frost heaving and subbase design.	Montana
Drainage and supporting power.	Nebraska
Fill construction. Compaction of subgrades.	Nevada
Low cost roads. Stage construction.	New Hampshire
Low cost road design.	New Jersey
Low cost roads.	New Mexico
Subgrades for concrete roads. Subbases.	New York
Low cost roads.	North Carolina
Subbase design. Treatment of failures on reconstruction.	North Dakota
Low cost roads. Drainage.	Ohio
Base courses. Selection of cherts, gravels and macadam aggregates.	Oklahoma
Fill construction. Warping of concrete pavements. Base courses.	Oregon
	Pennsylvania
	Rhode Island
	South Carolina
	South Dakota
	Tennessee
	Texas
	Utah
	Vermont
	Virginia
	Washington
	West Virginia
	Wisconsin
	Wyoming
+ Status Dec. 1932	Total
O Status Dec. 1934	Total

It was not until 1932, however, that the importance of densification in connection with the use of chemical admixtures began to be realized. Experiments performed at the Arlington laboratory of the Bureau of Public Roads disclosed that admixtures such as Portland cement and bituminous materials could best be distributed in fine-grained soils in the form of suspensions, solutions, or emulsions. The tendency of samples thus treated to become porous upon drying owing to the effect of the admixtures in reducing the shrinkage properties of the colloids suggested the necessity for mechanical compaction as part of the stabilizing procedure.

In 1889, Puchner (37) found that there is an optimum moisture content at which mixtures of soil and water exhibit a maximum penetrative resistance. Later work reported by Proctor (38) in 1933 furnished relations of moisture content and density of soil which could be utilized in embankment construction. Further work by the Bureau of Public Roads in 1934 on the effect of admixtures resulted in the formulation of a definite basis for determining the need for admixtures, the relative stabilizing effect of admixtures, and the construction requirements.

Field experimentation in the use of chemical admixtures which do more than stabilize the moisture content includes the use of Portland cement, bituminous materials, and the combination treatment with sodium silicate and calcium chloride to produce the calcium silicate precipitate.

Several sections of cement-stabilized soil road with bituminous-surface treatment were constructed by the South Carolina State Highway Department during 1934. A bituminous emulsion-stabilized soil road with surface treatment was constructed by the Virginia State Highway Department in 1935.

Soil Studies Used to Solve Road Problems.—California, under C. L. McKesson's direction, has utilized the lineal-shrinkage test as a guide in the selection of earth-fill materials since 1922, and Michigan has utilized the pedologist's survey methods since 1926. However, the year 1929 marks the beginning of the development of state soil laboratories along the lines of the newer procedures of soil testing and grouping.

New Hampshire was the first state to include such surveys as part of the routine preconstruction procedure (39). The reason was as much to prevent loss of life and limb as to preserve the integrity of constructed road surfaces. Here, as in several adja-

cent states, the subgrade soil changes in character very abruptly so that the change from soils that do not heave to those which do heave considerably, owing to frost, may cause the road surfaces suddenly to develop obstructions from several inches to almost a foot in height.

While such pavement distortions necessitated high maintenance costs, they were *far* more serious as traffic hazards. The heaving soils were found to have characteristics of two groups of the new classification, while the soils that did not heave were of another group, all easily distinguished in the field.

Consequently the new procedure afforded a simple yet accurate means of locating the portions of the subgrade requiring corrective measures to prevent heave.

A midwestern state—Missouri—was the second to establish a soils organization—also in 1929. The reason in this case was the failure of a fill constructed of poor material when good soil was readily available. The estimated expense for maintaining this one fill for several years after construction was deemed equal to that required to cover the cost of an adequate soils unit for a considerable number of years.

Minnesota followed next in an effort to prevent the loss of stability due to frost boils in roads located where silt soils abound (40). The discovery that the occurrence of high joints in concrete pavements was confined entirely to pavements constructed on soils of but two of the new groups was responsible for the fourth state's—Texas—establishing a soils organization.

As in the case of the first four states, soils units were established in other states because the new procedure offered a means of eliminating some particular difficulty. By 1932 soils units had been established by 13 states and the District of Columbia. The preceding chart, which gives the status of soil testing in the various state highway departments as of 1934, shows that the number then utilizing the new procedure had increased to about 30 with about a dozen more using soil information to at least some extent. The following outline shows the comprehensive scope of the field included in the subject of soil as an engineering material.

OUTLINE OF INVESTIGATIONS OF SOILS FOR HIGHWAY CONSTRUCTION

1. Standardization of definitions and terms.

- a. Materials: gravel, sand, caliche, shale, topsoil, etc.
- b. Fractions: coarse aggregate, fine aggregate, binder, silt, clay, etc.

2. Methods of subsurface exploration.
 - a. Resistivity measurements.
 - b. Soundings: rods, pipes, etc.
 - c. Borings: auger, wash, core.
 - d. Test pits.
3. Methods of sampling.
 - a. Disturbed state.
 - b. Undisturbed state.
 - (1) Natural soils: above or below water table.
 - (2) Stabilized soils.
4. Tests to determine properties of constituents exclusive of structure.
 - a. Mechanical analysis.
 - b. Chemical analysis: lime, silica, etc.
 - c. Physical tests: plasticity, shrinkage, etc.
 - d. Tests for pH value and adsorption.
 - e. Microchemical analysis: water-soluble chemicals.
5. Tests of stabilized soils to determine properties of constituents combined with effect of structure artificially produced by use of admixtures or manipulation, or both.
 - a. In place: stability, load distribution, etc.
 - b. Undisturbed samples in laboratory: capillarity, permeability, shear, etc.
 - c. Special tests of disturbed material: compactibility, compressibility, expansion, frost heave.
6. Tests for natural-foundation soils to determine properties of constituents combined with effect of natural structure or natural moisture content.
 - a. Undisturbed, in place: penetration, direct loading, etc.
 - b. Undisturbed samples in laboratory: compressibility, expansion, shear, etc.
 - c. Remolded in the laboratory.
7. Specifications for foundation soils located above the line of seasonal change; subgrades.
 - a. Character: grading, plasticity, capillarity, etc., to prevent detrimental
 - (1) Expansion due to water absorption.
 - (2) Heave due to frost.
 - (3) Settlement due to moisture loss.
 - (4) Deformations due to elasticity.
 - (5) Loss of stability.
 - (6) Landslides.
 - b. Method of construction:
 - (1) Excavation for cut.
 - (2) Compaction for fill.
 - (3) Preparation for subgrade.
8. Specifications for foundation soils located below the line of seasonal change: bridges, retaining walls, culverts, fills, and the like.
 - a. Character:

- (1) Load-compressibility relationship.
 - (2) Time-load-compressibility relationship.
 - (3) Elastic deformations and rebound.
 - (4) Stability.
 - (5) Effect of disturbance of natural structure.
- b. Design of bearing area:
 - (1) Footings on piles.
 - (2) Spread footings without piles.
 - (3) Rafts.
 - (4) Slopes of fills.
- c. Construction:
 - (1) Depth of excavation.
 - (2) Hydraulic or rolled method for fills.
9. Specifications for road surfaces, bases for surface treatment, and subgrades, comprised of graded soil or soil materials.
 - a. Materials:
 - (1) Natural material such as gravel, sand, shale, chert, and the like.
 - (2) Commercial products such as crushed stone, washed gravel, and the like.
 - (3) Special soil products such as limerock, caliche, disintegrated granite, and the like.
 - b. Proportioning of aggregate and binder.
 - c. Methods of construction.
10. Specifications for chemically stabilized materials for use in road surfaces, in bases for bituminous surface treatments, or in some special operation such as mud jacking.
 - a. Admixtures:
 - (1) Deliquescent chemicals.
 - (2) Crystallizing, flocculating, or deflocculating chemicals.
 - (3) Bituminous materials.
 - b. Grading and character of mineral aggregates and binder.
 - c. Proportioning of mineral and chemical constituents.
 - d. Methods of construction.

PART I
ORIGIN AND COMPOSITION OF SOIL

CHAPTER I

SOIL CONSTITUENTS

The soil is composed of particles differing physically in size, shape, and cellular structure, varying in chemical composition, and surrounded, at times, with water containing varying amounts of chemicals in solution. Organic matter, air, bacteria, and organisms are usually present. Soil particles may be coated also with adsorbed ions, as described later in Chap. IV.

Soil Terminology Not Standardized.—The lack of coordination in studies made by the different groups interested in soils has prevented the standardization of terms for either individual soil materials or different soil mixtures. Even those belonging to the same interested group have not as yet reached complete agreement as to the meaning of certain terms. In the highway field, for example, the term "clay" has been used to designate soil fines with a maximum size of 0.02 mm. in connection with sand clay and sand-clay gravel-road construction and with a maximum size of 0.005 mm. when used in connection with studies of sub-grade soils. Thus, Webster's definition of soil—"the loose surface material of the earth in which plants grow, in most cases consisting of disintegrated rock with an admixture of organic matter"—serves only as a generalization for the layman.

To the agronomist, according to Robinson (41), soil is "the superficial stratum of the regolith," consisting essentially of "mineral matter which has originated from rocks by the action of a series of weathering processes; organic matter, which has originated from the residues of natural vegetation and organic manures; soil moisture, containing substances in colloidal or in true solution; and soil air."

The regolith is defined as the fragmental debris mantling the rocks of the earth's crust. In the agricultural sense, therefore, the term soil is applied only to the upper portion, or topsoil, of the disintegrated rock mantle on the earth's surface. In the geologic sense, however, soil embraces as well the underlying rotted rock, or subsoil (41).

Geologists' Terminology for Rock Used.—Bedrocks as well as the subsoil are often included in the engineers' studies, in which case the geologists' terminology based on the origin of the rocks is used. Rocks may be of three kinds: the igneous rocks, made by the solidification of molten material; the sedimentary, or bedded rocks, formed from sediments deposited chiefly by water (and to some extent by air and ice); and the metamorphic rocks, formed by certain processes acting on preexisting rocks and altering their original characters to an extent such that the resultant rocks are considered as constituting a separate group.

Granite, diorite, rhyolite, andesite, and basalt are representatives of the igneous rocks; conglomerate, sandstone, shale, and limestone, of the sedimentary rocks; and gneiss, quartzite, slate, and marble, of the metamorphic rocks. According to Pirsson and Schuchert (42), "three-fourths of the land area of the globe is underlain by sedimentary rocks, and the other fourth by igneous and metamorphic rocks."

In mathematical computations made to determine its engineering properties, soil is assumed to consist of inert solids of varying size and constant volume and to contain pores which enlarge and shrink as the soil-mass changes in volume. The soluble materials, organisms, bacteria, etc., are not usually present in amounts sufficient to require consideration in the interpretation of the results of tests for physical properties. The character and amount of soluble soil constituents and of organic matter may, however, have an important bearing upon such problems as the durability of concrete and the selection of admixtures for use in the stabilization of soils.

The term "solid" is fairly well understood, although the ceramist (43) points out that "some apparently solid materials—especially certain glasses and undercooled fused materials—are more correctly described as liquids which are so highly viscous as to present the appearance of solids." Strictly speaking, solid substances are of a firm, compact nature, and each piece or portion of a solid has a definitely measurable length, breadth, and thickness which are independent of the support on which the solid is placed.

Soil solids have been classified on the basis of form and size of particle. Soil mixtures are classified on the basis of natural occurrence, mechanical grading, and physical properties.

Soil Solids Differ in Structure.—According to form, soil solids may be divided into two important classes, namely, crystalline and amorphous (43).

Crystalline substances consist of units of definite geometrical form or of fragments of such units. There are several forms. Perfect, regularly defined crystals occur in rocks that have cooled slowly from a molten state. Perfect crystals may also be produced when the liquid phase of a solution of one or more substances evaporates until the concentration of the liquid is too great for all of the substances to remain in solution. Such crystallization from solution occurs in nature in cavities in rock formations, and it is also the result of metamorphism. Quartz, mica, orthoclase, feldspar, dolomite, various limestones, and magnesite are among the refractory materials which occur in the form of perfect crystals.

Particles, or aggregates of particles, of irregular shape and exhibiting no outward appearance of being crystals may have an internal crystalline structure. This type of particle is most common in the compact rocks such as quartzite, gneiss, sandstone, magnesite, dolomite, and limestone.

A third form of crystalline solids is represented by "crystallites," "microliths," or "incipient crystals" (43) which are generally formed by the agency of heat and occur in nature in igneous rocks. They are not definitely crystalline but possess a more or less regular shape. Augite, hornblende, feldspar, etc., occur in rudimentary rocks in forms of this kind.

Amorphous solid substances have no definite geometric internal structure, although they may have a definite external form. They are distinguished from crystalline solids principally by the fact that while the latter, when crushed, still retain their definite structure, that of amorphous substances, being merely external, is destroyed when they are pulverized.

One type of amorphous solids is represented by certain substances, which, when cooling from a molten mass, have not crystallized but instead have formed isotropic, structureless, glassy masses similar in character to glazes on pottery.

Amorphous solids of the cellular type include substances such as the hardened volcanic-glass froth known as pumice and the diatomaceous earths which consist of the minute, siliceous skele-

tons of dead marine and fresh-water plants such as diatoms and microscopic animals such as the Radiolaria.

Other amorphous substances occur as irregular granules united to form larger masses or as stains or films on other grains. One of the commonest of such substances is dried or indurated clay. Many crystalline materials such as limestone, magnetite, and graphite may occur in the amorphous state as a result of abnor-

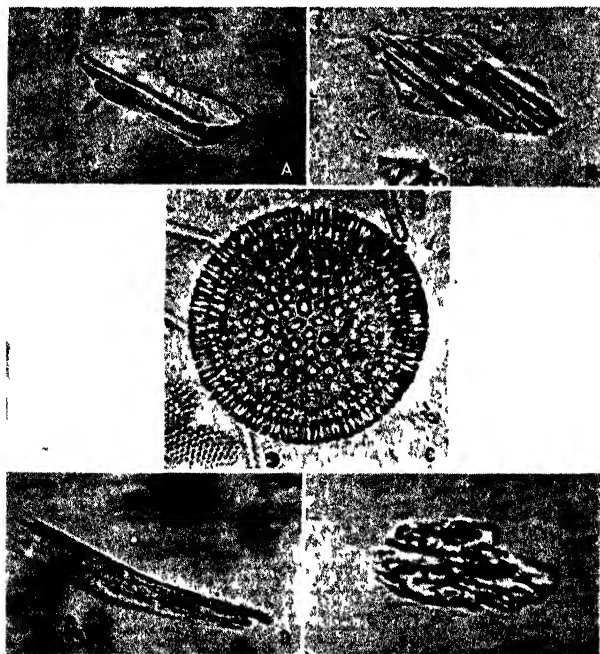


FIG. 1.—Photomicrographs of some common soil constituents. A, talc (relative magnification 1); B, pumice (r.m. 1); C, diatoms (r.m. 2.2); D, peat (r.m. 2.2); E, bentonite (r.m. 3.8).

malities in their mode of formation. The natural hydrous silicate of alumina of volcanic origin, known as bentonite, has been considered an amorphous material, although it is now known to contain crystalline matter also.

Photomicrographs of some common soil constituents are shown in Fig. 1.

Soil-fraction Names Based on Size of Particles.—The solids of which a soil is composed are classified by the Bureau of Public Roads according to particle size into six groups, as follows:

Gravel: particles retained on the No. 10 (2-mm.) sieve.

Coarse sand: particles passing the No. 10 sieve and retained on the No. 40 (0.42-mm.) sieve.

Fine sand: particles passing the No. 40 sieve and retained on the No. 270 (0.05-mm.) sieve.

Silt: particles between 0.05 and 0.005 mm. in diameter.

Clay: particles finer than 0.005 mm. in diameter

Colloids: particles finer than 0.001 mm. in diameter.

In early studies of sand-clay and topsoil roads, clay was used to designate particles with diameters less than about 0.02 mm.; silt, particles between 0.02 and 0.07 mm. (passing the No. 200 sieve); and sand, particles larger than 0.07 mm. (retained on the No. 200 sieve). In addition, the No. 60 (0.25-mm.) sieve was considered as separating the coarse from the fine sand. These size limits were purely arbitrary and were used because it was convenient to separate the particles by the method of mechanical analysis then in use.

These earlier size ranges have been supplanted by those now in use for several reasons:

1. Each of the present divisions represents a group of particles having a special physical significance, as explained more fully in a later discussion.

2. Use of the No. 40 sieve to separate coarse from fine sand eliminates one determination in the mechanical analysis, because tests to disclose the properties of the finer portion of soils are performed on the material passing the No. 40 sieve.

3. With the exception of the division between the coarse and fine sand, the present size limits correspond to those used by the Bureau of Chemistry and Soils, thereby facilitating the use of published information on soil surveys made by that bureau, in which the mechanical analysis plays an important part.

4. With present methods of mechanical analysis the grading according to the present sizes can be determined as readily as were the former sizes by the earlier methods.

Size Divisions Have Physical Significance.—Any crystalline mineral or rock material may be ground mechanically into particles having the sizes indicated by the six group divisions and, according to the size definitions, will then consist of sand, silt, clay, etc. In this case, however, the distinction between these fractions is entirely one of difference in the particle sizes, each fraction consisting of the same material in a different state of subdivision.

In natural soils, however, the distinction between the fractions is likely to be a matter of difference not only in size but also in form and composition, as is indicated by the following group descriptions:

Gravel.—The rock fragments which make up gravel are usually more or less rounded by water action and abrasion. The smaller pieces, up to about the size of baseballs, are known as pebbles, and the larger ones as boulders.

Since quartz is the hardest of the rock-forming minerals, it is the principal constituent of most gravels; and well-rounded

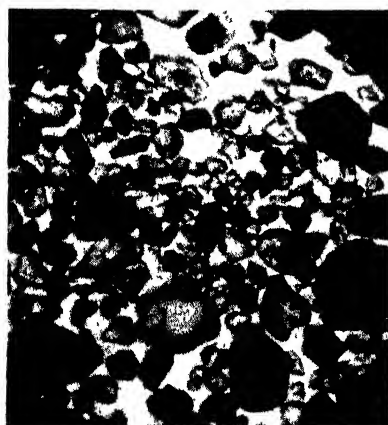


FIG. 2.—Photomicrograph of glacial sand.

pebbles and boulders, having undergone long wear, are usually composed almost entirely of quartz. On the other hand, gravel that is only slightly worn, and therefore rough and subangular, commonly includes other minerals such as granite, schist, basalt, or limestone.

Coarse Sand.—This fraction is usually rounded like pebbles and is likely to consist of the same minerals as the gravel with which it occurs (see Fig. 2).

Fine Sand.—The fine-sand particles are usually more angular than those of the coarse-sand fraction. This does not mean that the coarse sand has been exposed to water action and abrasion for a longer time than the fine sand but lies in the fact that the film of water between the particles serves as an effective buffer to protect the finer particles against abrasion.

Silt.—Silt usually consists of bulky grains, similar except for size to the fine sand and having the same mineral composition. In some cases, however, it may be largely a product of chemical decay rather than of rock grinding and as such may consist of silicates of aluminum and the alkaline earths and of oxides of iron. In other cases, the silt may be composed of foreign materials such as diatoms, pumice, or loess.

Clay.—While the coarser fractions consist usually and mainly of original fragments such as quartz and feldspar, the clay fraction consists almost entirely of the secondary products of chemical weathering. Clay was formerly considered an amorphous material, but more recent studies have shown it to be crystalline. The more plastic and less stable varieties are composed largely of flat, scalelike particles. Clay differs from the coarser fractions in that it is the chemically reactive portion of the soil, while the coarser fractions are inert.

Colloids.—In a strict sense soil colloids include only those finer clay particles which show pronounced Brownian movement when suspended in water. Searle (43) defines Brownian movement as the state of constant unordered motion, visible under the microscope, which is due to the fact that each particle possesses an electrical charge which causes it to repel other particles similarly charged.

According to Freundlich (44), Wiener arrived at the view that Brownian movement is a phenomenon such as would be demanded by the kinetic theory of heat; he considered the irregular impacts of the surrounding liquid molecules against the suspended particles to be the cause of the motion.

Freundlich (44) states that the theory of Brownian motion from the molecular-kinetic point of view was developed by Einstein and von Smoluchowski. The latter gave as the fundamental idea of his treatment that the mean kinetic energy of a particle that is moved by the blows of the surrounding molecules of liquid is, in a stationary condition, equal to that of the surrounding molecules; that is to say, it behaves as if it were a molecule of a dissolved substance. Einstein tacitly based his work upon the same view, by postulating that particles in Brownian movement exert the same osmotic pressure upon a wall impermeable to them as the same number of molecules would exert.

According to some authorities, clay particles up to 0.002 mm. in diameter exhibit Brownian movement in water, and this size therefore represents the upper limit of the colloidal clay. In testing soils for highway purposes, however, colloids are considered as consisting of particles 0.001 mm. in diameter and finer.

Composition of Solids Influenced by Climate.—The mineral constituents of the soil particles depend largely upon the chemical

composition of the parent rock and the conditions under which the soils were developed. In humid northern climates chemical weathering causes the removal of iron and aluminum oxides by leaching, resulting in soils containing relatively large accumulations of silica. Below the leached surfaces are found the clays and hardpans in which the weathering has not proceeded so far and in which, while the silica still predominates, the amounts of alumina and iron oxides are greater than in the surface soil. These weathered materials are underlain by the parent rock.

In humid tropical climates the silica is leached out of the soils with the result that the iron and aluminum oxides predominate. This type of weathering is termed lateritic and results in the formation of the lateritic soils of the South and the true laterites of the tropics.

The ratio of silica to the iron (ferric) oxide plus the aluminum oxide in soil is referred to as the "silica-sesquioxide ratio" and is designated by the symbol $\text{SiO}_2/\text{R}_2\text{O}_3$.

Clays high in silica (silica-sesquioxide ratio of 2 to 6 or 7), termed "pod sols," are acidic and consist principally of highly water-adsorbent, scalelike particles. The laterites (silica-sesquioxide ratio of 2 or less) are more nearly neutral and consist mainly of bulky or spherical particles.

With Wright's method of analysis (45), Joseph and Hancock (46) found that the $\text{SiO}_2/\text{R}_2\text{O}_3$ ratio of the material smaller than 0.002 mm. in a number of soils varied from 2.11 to 7.67.

In arid or semiarid climates the alkalies in the soil are not leached out completely, thus causing the soil to be neutral or alkaline. Other soil formations of equal importance, as well as detailed descriptions of the processes mentioned, are discussed by Keen (47).

Composition Influenced by Geographical Location.—Because of the effect of climate, soil solids of conspicuous chemical compositions are distributed geographically with a certain natural regularity in the same way as animals and plants. Just as any particular climatic belt is responsible for a definite flora and fauna, it is responsible also for soil types of definite chemical composition.

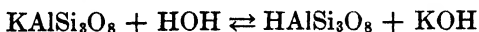
In Europe and Asia the isohyetal lines are more or less parallel to the isothermal lines; that is, as we move from the north southward, the temperature increases, and the rainfall decreases. There the soils are distributed parallel to the climatic belts, the

podsoils being found in the southern portion of the northern region and in part of the temperate region covered with conifers and deciduous forests. South of this region—in the temperate zone—the slightly podsolized type occurs. In the tropics with a high temperature and high rainfall the laterites are developed.

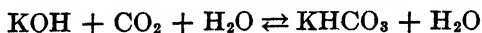
In the North American continent, especially in the eastern part, the direction of the isohyetal lines is, in general, perpendicular to the direction of the isothermal lines, and the geographic distribution of the zonal types does not follow the north and south direction as it does in Europe and Asia. In the humid northern portions of the United States are found the podsoils, and the laterites occur in the warmer humid southern regions. In the eastern parts the soils are acidic, and the alkaline soils are found in the arid western regions.

Composition Affected by Parent Rocks.—Aside from silica, iron, and alumina (48), the principal soil minerals are silicates, ferrosilicates, aluminosilicates, or ferroaluminosilicates of the common bases sodium, potassium, calcium, magnesium, and ferrous iron. Other bases, such as lithium, barium, or the heavy metals, may occasionally be present in appreciable amounts as may other types of silicates or other mineral salts.

The silicates or silica minerals are all somewhat soluble in water and, being salts of weak acids with strong bases, are greatly hydrolyzed. A convenient illustration is afforded by the well-known rock- and soil-mineral orthoclase, which with alteration becomes kaolin. Assuming its type of formula, the reaction of orthoclase with water may be represented by



Under ordinary soil conditions, with a relatively large proportion of carbon dioxide in the soil atmosphere, the potash formed would be more or less completely transformed to the bicarbonate:



Unless the hydrolysis of a silicate of the alkalis or alkaline earths is a reversible reaction, no silicate mineral could persist in the soil for any length of time, and all soils would soon become sterile wastes composed essentially of quartz, kaolin, and ferruginous oxides. It has been suggested also that the original mineral

particles are protected from decomposition by a jellylike coating on the surface of the soil grains.

Organic Matter (41).—The organic matter of the soil consists partly of recognizable fragments of plant material and partly of plant and animal material which has lost all traces of its original structure and has become changed into a dark, amorphous material called humus. While in some cases, such as in peat soils, the undecomposed or partially decomposed material may form a considerable portion of the total organic matter, usually 80 per cent or more consists of the decomposed, dark-colored, structureless humus.

The proportions of organic matter in soils varies considerably. While in certain tropical soils it may fall below 1 per cent, it may constitute practically the whole of the dry matter in peats.

Soil Water and Solutions.—If a soil is shaken up thoroughly with water, and the resulting solution filtered free of suspended matter and boiled to eliminate the carbon dioxide, the solution in the vast majority of cases will give an alkaline reaction with litmus. This is true also of the waters of most springs, ponds, or creeks.

But the mineral content of these waters varies widely. The water that passes down through the larger interstices of the soil is not long in contact with the individual soil particles and floccules, and because diffusion of dissolved mineral substances is quite slow, especially in dilute solutions, it takes up but little matter from such aqueous films as it may intercept.

A different condition exists with respect to that portion of the soil water which returns toward the surface by reason of capillary forces, to form the great natural nutrient medium for plants. This water is moving over the soil particles in films and with slowness. It is long in contact with successive fragments of any particular mineral and all the different minerals making up the soil. Consequently, it tends to become a saturated solution with respect to the mineral mass.

Many attempts have been made to extract the solution naturally existing in the soil and to analyze it. The results obtained have not been very satisfactory, owing mainly to the fact that the solution in a soil under suitable conditions for crop growth is held by a force of great magnitude—9,000 to 15,000 atmospheres (48).

By means of powerful centrifuges it has been possible to obtain small quantities—generally a few cubic centimeters at a time—of dilute solutions—about 6 to 8 p. p. m. of solution for phosphoric acid (P_2O_5) and 25 to 30 p. p. m. for potash (K_2O). The analysis

TABLE 1.—COMPOSITION OF ASH OF ORDINARY CROPS
In per cent

Crops	Potash and soda alkalis	Mag- nesia	Lime	Phos- phoric acid	Silica	Sul- phuric acid	Chlo- rine
Cereals.... { grain..	30	12	3	46	2	2.5	1
{ straw..	13 to 27	3	7	5	50 to 70	2.5	2
Legumes... { kernel	44	7	5	35	1	4	2
{ straw..	27 to 41	7	25 to 39	8	5	2 to 6	6 to 7
Root crops { roots..	60	3 to 9	6 to 12	8 to 18	1 to 4	5 to 12	3 to 9
{ tops..	37	3 to 16	10 to 35	3 to 8	3	6 to 13	5 to 17
Grasses in flower	33	4	8	8	35	4	5

TABLE 2.—COMPOSITION OF RIVER WATERS
In parts per million

River	Potash and soda	Mag- nesia	Lime	Phos- phoric acid	Silica	Sul- phuric acid	Chlo- rine
Yukon, Alaska.....	8	7	30	8	9	0.4
St. Lawrence, Pointe des Cascades.....	8	10	45	Trace	33	48	2
Missouri, Montana....	32	18	58	0.22	19	22	18
Mississippi, near Car- rollton, La.....	20	41	41	9	16	10
Rio Grande, Fort Craig, N. M.....	44	2	23	10	47	36

of a few cubic centimeters of a very dilute solution is in itself very difficult, involving necessarily more or less uncertainty as to the absolute value of the results.

Wheat, corn, and some common grasses have been grown to satisfactory maturity in tap water with a concentration of 7 p. p. m. of potassium and 0.5 p. p. m. of phosphoric acid. In

TABLE 3.—WATER ANALYSES

From the laboratory of the U. S. Geological Survey, *Water Supply Paper 364*,
by F. W. Clarke, 1914

State	Sample number	K + Na	Mg	Ca	CO ₂	SiO ₂	SO ₄	Cl
Parts per million								
Maine	1	27	16	136	27	17	370	Trace
New York	2	102	30	180	1	8	293	206
District of Columbia	3	6	1	3	25	8	10
Virginia	4	10	3	4	None	33	11	7
Virginia	5	84	15	524	159	21	1,287	10
Virginia	6	22	35	133	223	24	129	3
North Carolina	7	29	9	151	78	30	333	3
South Carolina	8	1,055	8	14	40	36	Trace	944
Florida	9	74	17	80	15	52	108
Mississippi	10	132	74	198	75	985	72
Tennessee	11	8	2	12	22	13	1
Kentucky	12	481	19	21	10	187	522
Illinois	13	300	447	542	113	14	3,004	35
Illinois	14	681	277	427	1,142	12	1,695	24
Iowa	15	19	32	76	219	Trace	None	1
Missouri	16	3	3	43	15	4	2
Arkansas	17	4	Trace	69	12	8	3
Oklahoma	18	293	35	86	121	24	32	482
Wyoming	19	20	3	9	42	8	9
Wyoming	20	197	63	236	50	508	160
Montana	21	30	44	168	248	29	222	12
Colorado	22	6,525	7,285	302	28	41,362	1,492
Colorado	23	261	33	58	10	514	None
Milligrams per liter								
West Virginia	24	4,750	380	2,220	Trace	None	11,900
Texas	25	31,380	290	2,000	3,510	53,070
Wyoming	26	1,424	406	71	67	14	4,129	58
Utah	27	8	13	43	97	7	11	5
Utah	28	7,083	93	1,143	144	46	218	13,703
Nevada	29	32	4	23	56	23	29	13
California	30	11	3	9	28	14	5	2
California	31	670	60	59	578	313	227
Oregon	32	15,228	6,006	232	706	13,462
Washington	33	10,504	108	Trace	6,419	113	4,362	3,526
Gulf of Mexico	34	11,596	1,305	442	126	2,742	20,076

Number

Identity of Samples

1. Water from spring near Mount Mica, Paris, Me.
2. Water from Caledonia spring, Caledonia, N. Y.
3. Well at Corby's bakery, 71 ft. deep, Washington, D.C.
4. Well near Washington Brick Company's plant, Alexandria, Va.
5. Spring in Loudon County, Virginia.
6. New Hot Spring, temperature 99°F., Virginia Hot Springs, Va.
7. Spring near Towesville, N. C.
8. Wentworth Street well, Charleston, S. C.
9. Surface drainage water, St. Augustine, Fla.
10. Well near Clinton, Hinds County, Mississippi.
11. Spring water, Mountain City, Tenn.
12. Murray well, near Frankfort, Ky.
13. Spring at Leansboro, Ill.
14. American Carlsbad Spring, Nashville, Ill.
15. Artesian well, Story City, Iowa.
16. Spring near Thurman, Mo.
17. Mountain Valley Spring, near Magnet Cave, Ark.
18. Pavilion Spring, Sulphur, Okla.
19. The Yellowstone Lake, Wyo.
20. Hot River, Wyo.
21. Livingston Warm Springs, Mont.
22. Spring near Denver, Colo.
23. Fariss artesian well, Pueblo, Colo.
24. Deep well near Coalburg, W. Va.
25. Brine from Humble oil field near Houston, Tex.
26. Lake De Smet, Wyo.
27. Bear River, a tributary of Great Salt Lake.
28. Utah Hot Springs, 8 miles north of Ogden, Utah.
29. Walker River, Nev.
30. Lake Tahoe, Calif.
31. Warm Spring, Mono basin.
32. Albert Lake, Ore.
33. Soap Lake, Wash.
34. Sea water from Gulf of Mexico, near Tortugas.

this connection investigators believe that the ratio of potassium to phosphoric acid of 14 to 1 is quite important. Wheat, grasses, cowpeas, vetches, potatoes, and other plants have grown satisfactorily in solutions made by shaking up a soil in distilled water and separating the solution from the solid particles by means of an unglazed porcelain filter.

The soluble constituents of soils are revealed by analyses of plant ashes and of waters from springs, lakes, and rivers.

Analyses of ashes of ordinary crops are given in Table 1 (49). In presenting these data the authors call attention to the fact that plant physiologists and soil scientists have been and still are divided into several schools. One group maintains that plants do not dissolve significant quantities of minerals that would other-

TABLE 4.—WATER-EXTRACTED CHEMICALS FROM SOILS
In parts per million

Soil	Potash and soda	Mag- nesia	Lime	Phos- phoric acid	Silica	Sul- phuric acid	Chlo- ride of sodium	Car- bonic acid
Bonn, Saxony	126	38	128	31	48	100	59	
Chemnitz, Saxony	38	37	84	Trace	26		48	
Sassafras, sandy loam	13	18	74	7	6	54		14
Norfolk, N C, sandy soil	21	23	58	10	8	43		20
Janesville, Wis, loam	25	52	135	17	40	125		29
Hagerstown Pa clay loam	22	77	165	12	21	188		97

wise remain undissolved; another school emphasizes the important role that the root exudations exercise in this connection (50).

Table 2 from Hilgard (51) contains data on the composition of river waters. Additional analyses are given in Table 3. Table 4, also from Hilgard, affords some insight into the actual and possible solvent effects of water in the soil.

Authorities agree that it seems impossible to exhaust a soil's solubility by continuous leaching with water. This was demonstrated in 1863 and 1864 by Ulbricht (52) and by Schulz (53); their general conclusions were corroborated by King (54) in 1904.

King's first leachings were made by shaking up the soil with ten times its dry weight of water for 3 min. The ingredients of the filtrates were then determined by very delicate, mostly colorimetric methods. The soil was dried at 120°C. between successive leachings. Hilgard found that at each drying not only are

the soluble minerals again drawn to the surface but heating a soil renders additional amounts of soil ingredients soluble both in water and in acids.

Problems

1. What are the principal constituents of soil?
2. Give a definition of soil.
3. State the three groups into which rocks are classified and name an example of each group.
 4. (a) How are soil solids classified?
 - (b) Name the two important forms of soil particles and give the essential characteristics of each. How are these forms distinguished from each other?
 - (c) Name the groups of soil solids according to particle size and give the distinguishing characteristics of each group.
5. What is meant by silica-sesquioxide ratio?
6. (a) How are podsol soils formed?
 - (b) How are lateritic soils formed?
 - (c) How does a podsol soil formed under arid conditions differ from one formed under humid conditions?
7. Name the common mineral-forming bases found in soils.
8. What effect does the reversible chemical reaction taking place during the hydrolysis of a silicate of the alkalies or alkaline earths have upon a soil?
9. Discuss the difference in action with respect to dissolved mineral substances between water percolating through the larger interstices of the soil and the water moving through the soil by capillarity.

CHAPTER II

SOIL MIXTURES

Soil mixtures may be defined as combinations of soil solids and air; of soil solids and water or solutions; and of solids differing in size, chemical characteristics, and shape. They may be grouped with respect to constituents and performance. In this chapter disturbed soil mixtures are discussed, as contrasted with soil mixtures in natural state discussed later in Chap. III.

General Terminology Listed.—Different symbols for the same terms, in some cases, and the same symbols for different terms, in other cases, have been used in the variety of literature from which the composite information on soils has been obtained. The use of the letter e to represent both the base of the Naperian system of logarithms and the voids ratio of soil mixtures is an illustration of this type of duplication. The Bureau of Public Roads is attempting to eliminate the confusion likely to be caused by such duplication by standardizing some of the symbols used to represent the more common terms employed in soil investigations. The nomenclature illustrative of these efforts used in this text differs, therefore, in some respects from that used in publications listed in the bibliography. A list of the more general terms used is given in Table 5. A more complete list, arranged by chapters is given in Appendix II.

Density Depends upon Porosity.—Relations of pores and solids in the soil mass are used in mathematical computations to indicate the density of the soil, which is defined as the weight of a substance per unit of volume. In the c.g.s. (centimeter-gram-second) system density is expressed in grams per cubic centimeter, and in the f.p.s. (foot-pound-second) system in pounds per cubic foot. Mathematical computations involved in soil mechanics are simplified by use of the c.g.s. system because of the fact that for practical purposes 1 cc. of water weighs 1 gram, thus causing the density of water to be 1 in the c.g.s. system as compared with 62.4 in the f.p.s. system. However, 1 kg. per square centimeter is approximately equal to 1 ton per square foot (2,046 lb. per square

foot). Consequently these terms are used synonymously in discussions of load or pressure.

Any material, like soil, which consists of solids and pores has two densities: that of the solids, termed "absolute" density, and

TABLE 5.—GENERAL NOMENCLATURE LISTED

<i>a</i>	Area of water column—pores in soil or standpipe in permeameter.
<i>A</i>	Area of soil mass—loaded area, drainage face.
<i>B</i>	Constant of semilogarithmic relations (moisture content or voids ratio at 1 unit of load).
<i>c</i>	Coefficient of cohesion and coefficient of consolidation.
<i>C</i>	Centigrade.
<i>d</i>	Thickness of soil samples.
<i>D</i>	Thickness of soil strata.
<i>e</i>	Voids ratio—base of natural logarithms.
<i>E</i>	Porosity.
<i>F</i>	Fahrenheit.
<i>g</i>	Gravitational constant.
<i>G</i>	Specific gravity.
<i>h</i>	Pressure head and height of capillary rise.
<i>J</i>	Exponent in soil consolidation computations.
<i>k</i>	Coefficients of flow—water and heat.
<i>L</i>	Length of area.
<i>m</i>	Width of area.
<i>n</i>	Coefficient of viscosity.
<i>N</i>	Number in series, or normal pressure.
<i>p</i>	Pressure.
<i>P</i>	Load.
<i>Q</i>	Discharge—water or heat, and settlement of soil.
<i>r</i>	Radius.
<i>R</i>	Metallic ions.
<i>s</i>	Diameter of soil particles, unit shear.
<i>S</i>	Shear strength.
<i>t</i>	Duration of time of load or flow.
<i>T</i>	Temperature.
<i>v</i>	Velocity.
<i>V</i>	Volume.
<i>w</i>	Moisture content.
<i>W</i>	Weight.
<i>Z</i>	Constant of semilogarithmic relations (change in <i>w</i> or <i>e</i> for change of unity in logarithm of load).

that of the mass, termed "bulk" density. The general term "density" is used when mass or bulk density is meant.

The ratio of the density of any substance to that of water is termed the specific gravity of the substance. Soil has also two specific gravities: absolute specific gravity, that of the solids; and

bulk or apparent specific gravity, that of the mass. In this case, however, the general term specific gravity means absolute specific gravity.

Computation of Density Illustrated.—The density of soils is indicated by such terms as porosity, pore or voids ratio, and moisture content. The porosity E is the volume of the pores expressed in percentage of the bulk volume (pores plus solids). The pore or voids ratio e (see Fig. 3) is the ratio of pore or void volume to that of the solids. The moisture content w is the weight of mois-

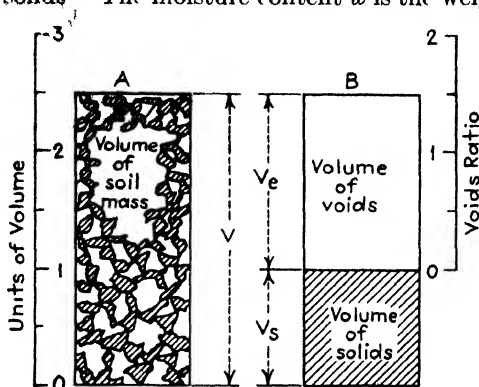


FIG. 3.—Diagram illustrating pores (voids) and solids in the soil mass.

ture in the pores expressed as a percentage of the weight of dried solids.

In completely saturated soils, then, when G = the specific gravity, e , as defined, $= \frac{E}{100 - E}$
from which

$$E = \frac{e}{1 + e} \times 100 \quad (1)$$

and

$$e = \frac{w}{100} \times G \quad (2)$$

This is because

$$e = \frac{\text{cubic centimeters of water}}{\text{cubic centimeters of solids}}$$

and

$$w = \frac{\text{grams of water}}{\text{grams of solids}} \times 100$$

One cubic centimeter of water, as noted above, weighs 1 gram; and 1 cc. of solids weighs G grams. Therefore,

$$w = \frac{\text{cubic centimeters of water}}{G \times \text{cubic centimeters of solids}} \times 100$$

A sample of muck soil is used for illustrative purposes in connection with the mathematical relationships discussed in this book. It has constants as follows:

Specific gravity $G = 2.71$

Volume of solids $V_s = 1.0$ or unity

Volume of pores $V_r = 1.5$

Then

Voids ratio $e = V_r/V_s = 1.5/1.0 = 1.5$

Volume of soil mass per unit of soil volume

$$V = 1 + e = 1.0 + 1.5 = 2.5$$

$$\text{Porosity } E = \frac{e}{1 + e} \times 100 = \frac{1.5}{2.5} \times 100 = 60 \text{ per cent}$$

Moisture content w required to fill the pores =

$$\frac{e \times 1.0}{1.0 \times G} \times 100 = \frac{100e}{G} = \frac{150}{2.71} = 55.4 \text{ per cent}$$

Voids ratio $e = 2.71w/100 = 2.71 \times 55.4/100 = 1.5$

Absolute density $D_s = 2.71$ in the c.g.s. system, and $2.71 \times 62.4 = 169.1$ in the f.p.s. system.

When the pores contain no water,

Bulk specific gravity,

$$G_0 = \frac{V_s \times G}{V} = \frac{1.0 \times 2.71}{2.5} = 1.084$$

Bulk density, or density, $D_b = 1.084$ in c.g.s. units, and $1.084 \times 62.4 = 67.64$ in f.p.s. units.

When the pores are filled with water,

$$\text{Bulk specific gravity} = \frac{1.0 \times 2.71 + 1.5 \times 1}{2.5} = \frac{4.21}{2.5} = 1.684$$

Bulk density = 1.684 in c.g.s. units and $1.684 \times 62.4 = 105.1$ in f.p.s. units.

However, when hydrostatic uplift is fully active, as would be the case if the sample were completely immersed, the following values would be used in computations:

Absolute density = $2.71 - 1.0 = 1.71$ in c.g.s. units and $1.71 \times 62.4 = 106.7$ in f.p.s. units.

Bulk specific gravity = $1.0 \times 1.71/2.5 = 0.68$

Bulk density = 0.68 in c.g.s. units, and $0.68 \times 62.4 = 42.4$ in f.p.s. units.

Geological Designation of Soils.—Soil mixtures may be termed “residual” or “transported” depending on whether they have remained *in situ* or have been moved from locations where they had originally been found. Residual soils include all deposits derived by the process of rock weathering or from organic accumulation in place. They include deposits derived from the decay of the immediately underlying rocks and of cumulous deposits which have been formed in place from the accumulation of organic matter with ordinarily small amounts of rock waste, such as many of the peat and muck deposits in ponds and lakes (55).

Often there is little similarity in composition between the soil and the underlying rock, thus indicating that agencies other than weathering have been at work. Residual soil, which is naturally the first to be formed, is often transported by water, wind, or glacial action to other localities and there laid down in distinct layers. Because of the mixing action of these agencies these layers often contain many more minerals than are present in the underlying rock. Even in beach sand many minerals other than quartz can usually be found. These agencies have been at work for so long that practically every soil contains all the common rock-forming minerals.

The transporting agency determines, to a large extent, the size, shape, and arrangement of the individual particles comprising the soil layer. Thus soils deposited by water include sedimentary clays, mucks, and silts (alluvium deposits) and the sands and gravel found in stream beds. Glaciers have carried soil materials and deposited them when they melted. Such deposits are called “moraines,” and the material comprising them is called glacial till, or boulder clay. In addition, the glaciers have been responsible for the formation of the inactive rock powders. Wind-borne deposits, termed aeolian, include the blow sands and the loess soils.

Designations Based on Grading.—The sizes of the individual solids comprising a given soil mass may differ widely owing to

the variations in the rate of weathering and because of the mixing effect of the transporting agencies. The term "texture" indicates the particle-size distribution for a given soil. Thus when we speak of a sandy soil, what we actually mean is that there is a predominance of large particles, but, in all probability, there will be present varying percentages of grains of many

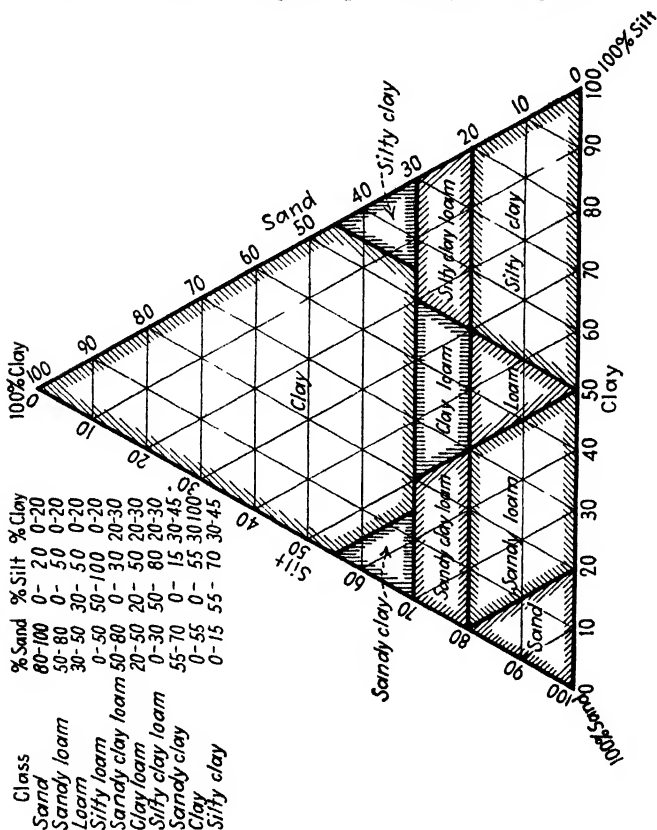


Fig. 4.—Textural-classification chart.

sizes including the very fine clays. The particle-size distribution or, in other words, the grading is determined by the mechanical analysis. Mixtures of specific gradings are designated as distinct soil types with names such as sands, clay loams, silt loams, etc. Sandy materials are designated as "light-textured" and clays as "heavy-textured" soils. The chart (Fig. 4), developed by A. C. Rose of the U. S. Bureau of Public Roads,

shows the mechanical grading of the 10 types classified according to texture.

Considerable confusion has resulted from the fact that the terms "sand" and "clay" have been used to designate fractions of particular size in the particle-size classification as well as types of soil mixtures in the textural classification. To avoid this misunderstanding, the terms "sand" and "clay" are used in this book as names of fractions consisting of solids of given size. The term "sands," "granular soils," or "sandy soils" is used instead of "sand"; and the term "clays," "clay soils," or "binder soil" instead of "clay" to designate soil types. Also, the term "loam" as used here does not imply the presence of considerable decomposed organic matter according to the popular understanding of the term.

Terms Based on Texture.—Terms descriptive of texture are defined as follows:

Sandy Soil.—Sandy soil is loose and granular. The principal individual grains can readily be seen or felt. Squeezed in the hand when dry, it will fall apart when the pressure is released. Squeezed when moist, it will form a cast but will crumble when touched.

Sandy Loam.—A sandy loam is a soil containing much sand but having enough silt and clay to make it somewhat coherent. The individual sand grains can readily be seen and felt. Squeezed when dry, it will form a cast which will readily fall apart; but if squeezed when moist, a cast can be formed which will bear careful handling without breaking.

Sands and sandy loams are classed as coarse, medium, fine, or very fine, depending on the proportion of the different-sized particles that are present.

Loam.—A loam is a soil having a relatively even mixture of the different grades of sand and of silt and clay. It is mellow with a somewhat gritty feel yet fairly smooth and slightly plastic. Squeezed when dry, it will form a cast which will bear careful handling, while the cast formed by squeezing the moist soil can be handled freely without breaking.

Silt Loam.—A silt loam is a soil having a moderate amount of the fine grades of sand and only a small amount of clay, over half of the particles being of the size called "silt." When dry it may appear quite cloddy, but the lumps can be readily broken,

and when pulverized it feels soft and floury. When wet the soil readily runs together and puddles. Either dry or moist it will form casts which can be freely handled without breaking. If squeezed between thumb and finger, it will not "ribbon" but will give a broken appearance.

Clay Loam.—A clay loam is a fine-textured soil which breaks into clods or lumps which are hard when dry. When the moist soil is pinched between the thumb and finger it will form a thin ribbon which will break readily, barely sustaining its own weight. The moist soil is plastic and will form a cast which will bear much handling. When kneaded in the hand it does not crumble readily but tends to work into a heavy, compact mass.

Clay Soil.—Clay soil is a fine-textured soil which forms very hard lumps or clods when dry. When the moist soil is pinched out between the thumb and fingers it will form a long, flexible ribbon.

Gravelly or Stony Soils.—All of the preceding classes of soils, if mixed with a considerable amount of gravel or stone, may be classed as gravelly sandy loams, gravelly clays, etc.; as stony sandy loams, stony loams, etc.; or as sandy clay loams, sandy clay soils, etc.

Terms Based on Composition and Structure.—Among other terms commonly used in descriptions of soil mixtures are names of materials such as gumbo, adobe, mud, dust, blow sand, kaolin, marl, talc, chalk, lime rock (as distinguished from limestone), and caliche; terms indicative of chemical composition such as peat soil, muck soil, alkaline soil, saline soil, calcareous soil, and acid soil; designations of mixtures of soil and water, such as pastes, slurries, and colloidal state; and terms descriptive of structure such as natural, remolded, dried, and powdered. They are defined as follows:

Gumbo.—A class of peculiar, fine-grained soils usually devoid of sand and rich in alkaline compounds. When saturated with water they become impervious and waxy or soapy in appearance and to the touch.

Adobe.—Heavy-textured alluvial clay soil from which unburnt brick are made. Occurs mainly in the arid regions of the southwestern United States.

Mud.—A slimy or pasty mixture of earth and water or of volcanic ashes and water; mire.

Dust.—Fine, dry particles of earth or other matter so pulverized that they may be lifted and carried by the wind.

Blow Sand.—Wind-borne, free-moving dune sand.

Kaolin.—A very pure white clay used to form the paste of porcelain; china clay; it is a hydrous silicate of alumina, $H_4Al_2Si_2O_9$, derived from the decomposition of feldspar. Often colored by iron compounds.

Marl.—An indefinite term applied to any earthy, crumbling deposit containing varying quantities of calcium carbonate, clay soil, sand, and carbonaceous material.

Talc.—A soft mineral of soapy feel; an acid magnesium metasilicate, $H_2Mg_3(SiO_3)_4$.

Lime Rock.—An unconsolidated or partially consolidated form of limestone, usually containing shells or shell fragments. Of marine origin.

Chalk.—A soft, porous variety of lime rock, consisting in some cases of microscopic shell fragments and in others of chemically precipitated particles of calcium carbonate.

Caliche.—A term applied in the United States to a group of formations consisting chiefly of calcium carbonate and silica, in the form of clays, sands, and gravels, cemented into a conglomerate by calcium carbonate deposited through the evaporation of ascending or descending ground waters. The term caliche was originally applied to the nitrate deposits of Chile, which are somewhat similar in appearance to the caliches of the United States.

Peat Soil.—Composed predominantly of organic material, highly fibrous, with easily recognized plant remains.

Muck Soil.—Composed of thoroughly decomposed black organic material, with a considerable amount of mineral soil material, finely divided and with a few fibrous remains.

Alkaline Soil.—A soil containing an excessive amount of the alkaline (in true chemical sense) salts.

Saline Soil.—A soil containing excessive amounts of the neutral or nonalkaline salts.

Calcareous Soil.—A soil containing sufficient calcium carbonate to effervesce when tested with weak (0.1 *N*) hydrochloric acid. Depending on the amounts present, these soils may be designated as slightly calcareous, strongly calcareous, etc.

Acid Soil.—A soil deficient in available bases, particularly calcium, and giving an acid reaction when tested by standard methods.

Additional terms for the engineer are defined in Appendix III.

Designation According to State.—Soils may be plastic or friable, depending upon the cohesion between the solid and liquid particles when the soils are mixed with water. Plastic soils, termed also “cohesive,” include most of the natural soils. There is considerable cohesion between the solid particles and liquid, so that mixtures of plastic materials and water have the property of forming true pastes with very definite characteristics such as plasticity and flow under pressure. Upon evaporation of moisture such pastes harden into compact masses.

Plastic materials may be defined as those which have the property of deforming under a constant shearing stress. They are contrasted with elastic materials in which the stress must increase with increase of strain. The term “plastic” used to describe soils, however, does not mean that the soil is a plastic material as defined above but that the soil at certain moisture contents may form a paste which has the properties of plastic materials.

Friable soils, termed also “cohesionless,” are the converse of plastic soils. Quartz ground in a ball mill to the fineness of clay has the properties illustrative of the friable materials. Such materials when mixed with water do not form true pastes and upon evaporation of the moisture become increasingly easy to crumble until, when thoroughly dry, they will fall to powder at a slight touch.

A slip or slurry is a mixture of solid and liquid in which the liquid predominates. A slip, like a liquid, takes the shape of the vessel into which it is placed. It also flows like a liquid and does not require any pressure to deform it, whereas a solid or a paste will not flow but changes its shape only when subjected to pressure. Slips may be made from friable as well as plastic soils and water.

Pastes are physical mixtures of one or more solids and liquids with definite characteristics such as plasticity and flow under pressure.

The colloidal state may be defined (43) as a physical condition of matter consisting of at least two parts or phases: (a) the

colloidal particles termed the disperse phase; and (b) the dispersion medium in which the colloidal particles are suspended or distributed. In contrast with true solutions colloidal states have only a slight influence on the vapor pressure, freezing point, and boiling point of the dispersion medium.

The chief colloidal systems are solutions and suspensions (solids suspended in liquid) and emulsions (liquids suspended in liquid). The solutions may be divided into "sols" and

When the suspended particles are so small that they remain indefinitely long in suspension the colloidal state is termed a "colloidal sol" as distinguished from ordinary suspensions from which settlement or deposition of the suspended particles occurs rapidly.

There has been the suggestion (43) that particles larger than 0.0001 mm. may be regarded as forming "coarse suspensions," those between 0.0001 and 0.000001 mm. "sols," and those smaller than 0.000001 mm. as forming molecular solutions.

Gel Formation.—In a sol the electrical charges of the particles in suspension are of the same sign. If particles of opposite sign are introduced, the two groups of oppositely charged particles are rapidly attracted to each other and form "flocs" too large to remain in suspension. They gradually settle to the bottom of the vessel to form a "coagulum." A gel is the deposit or sediment formed when the suspended matter of a sol is coagulated.

Table jellies are typical gels, but often clay pastes are also in the gel state. There is the possibility that flint and chert are amorphous colloidal gels formed by the coagulation of colloidal sols, the coagulum being afterward dehydrated and possibly hardened by pressure and other influences.

Sediments Formed from Suspensions.—Sand, silt, and clay may be suspended in water long enough to be carried considerable distances. It has been estimated that 340,500,000 tons of soil is transported in this way every year by the Mississippi River. More than one-third as much material (136,400,000 tons) is carried in solution. The estimated dissolved solids transported annually into the oceans is nearly 2,735,000,000 metric tons (1 metric ton = 2,204 lb.). The Mississippi carries about 136,000,000; the Connecticut, a small river, about 1,000,000; the Danube, more than 22,000,000; and the Nile, nearly 17,000,-

000 tons (42). Eventually, the suspended solids settle out to form sediments and in this way differ from the minute colloids which in the disperse state can be expected to remain in suspension indefinitely.

The process of sedimentation in all types of suspensions is controlled by the same basic physical laws. This is true for dust, which is a suspension of soil particles in air, as well as for clouds or fog, which are suspensions of moisture particles in air.

In all cases gravity exerts an attraction on the suspended particles, termed the suspensoids, which produces a tendency for them to move downward. Resisting this tendency is the buoyant effort of the suspending medium. The first requirement for sedimentation therefore is that the weight of the suspensoids exceed the buoyant effort of the suspending medium. This requires that the former have higher specific gravity than the latter.

The speed of settlement depends upon the tendency of the suspensoids to move downward and the resistance to such movement furnished by friction between the suspensoids and the suspending medium. The more the density of the suspensoids exceeds that of the suspending medium the greater becomes the motivating influence. The retarding friction depends upon the viscosity of the suspending medium. Its magnitude is expressed by the coefficient of viscosity which is defined as the tangential force required to move a unit area of a plane surface with unit speed relative to another parallel plane surface from which it is separated by a layer of the substance having unit thickness. The c.g.s. units of viscosity are dyne-seconds per square centimeter, termed "poises."

The greater the viscosity of the medium and the more irregular the shape and the rougher the surfaces of the suspensoids the greater will be the friction, and the slower will be the speed of sedimentation. The retarding friction increases with increase in the surface area of the suspensoids, and the motivating tendency with increase in their weight. As the size of particle decreases, the surface area increases at a much faster rate than the weight. Consequently, with other variables remaining constant, the smaller the suspensoids the slower will be their rate of settlement.

It can be readily shown, by applying the rule that the surface of a sphere is equal to 3.1416 times the square of its diameter,

that the total interstitial surface of a cubic foot of sand composed of grains 1 mm. in diameter is about 1,000 sq. ft.; that of a cubic foot of silt composed of grains 0.02 mm. in diameter, about 50,000 sq. ft., or more than 1 acre; and that of a cubic foot of material composed of grains only 0.001 mm. in diameter, about 1,000,000 sq. ft., or more than 20 acres. From experimental data King calculated the aggregate surface of a cubic foot of ordinary loam soils to be about 1 acre and of a cubic foot of fine clay soils to be about 4 acres.

If a stone is dropped into water, it sinks rapidly. When ground into fine particles, however, the gravitational force exerted on each particle is proportionately small, while the surface in contact with the water and the resultant friction are relatively great. Such particles soon reach their maximum velocity and thereafter fall at a continuously uniform rate proportional to their size.

Speed of Particle Settlement.—The mathematical relationship between the size and settling rate of spherical bodies in a suspending medium, as deduced by Stokes (56), may be expressed as follows:

$$s^2 = \frac{30n}{g(G - G_1)}v \quad (3)$$

in which s = diameter of particle, millimeters.

n = coefficient of viscosity of suspending medium, poises.

g = gravitational constant (980).

G = specific gravity of particle.

G_1 = specific gravity of suspending medium.

v = settling velocity in centimeters per minute.

Some conception of the effect of size of particle upon the speed of sedimentation is furnished by means of formula (3) with the substitution of assumed numerical values as follows:

n = 0.0102 poise, viscosity of water at 67°F.

G = 2.65, specific gravity of average soil solids

G_1 = 0.9984, density of water at 67°F.

Then

$$s^2 = \frac{30 \times 0.0102 \times v}{980(2.65 - 0.9984)}$$

and •

$$v = 5,290s^2 \text{ cm. per minute}$$

from which

$$v = 173s^2 \text{ ft. per minute}$$

According to this, the times required for spheres representative of the various soil-particle sizes to settle from the surface to the bottom of a lake 20 ft. deep would be as follows: coarse-sand grain with diameter of 1.0 mm., 6.9 sec.; fine-sand grain, 0.1 mm., 11.6 min.; silt grain, 0.01 mm., 19.3 hr.; clay particle, 0.001 mm., 80 days; and colloids, 0.0001 mm., almost 22 years.

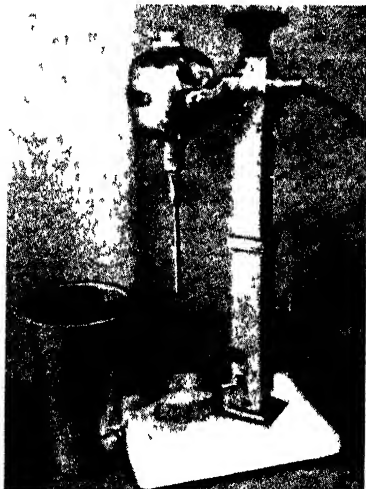


FIG 5.—Stirring device used in dispersing soils for hydrometer analysis.

Slow settlement of particles in air has also been observed. According to Udden (1915), wind will carry sand grains distances that vary roughly as follows: 8 to 1 mm., several feet; 1 to $\frac{1}{4}$ mm., a few rods; $\frac{1}{4}$ to $\frac{1}{8}$ mm., less than a mile; $\frac{1}{8}$ to $\frac{1}{16}$ mm., several miles; $\frac{1}{16}$ to $\frac{1}{32}$ mm., 200 miles; $\frac{1}{32}$ to $\frac{1}{64}$ mm., 1,000 miles; $\frac{1}{64}$ mm. and less, around the world.

Size of Solids Determined by Speed of Settlement.—A simple procedure for determining the distribution of particles of different sizes in a soil mixture is to determine the grading of particles larger than 0.05 mm. by sieve analysis and that of smaller particles by a hydrometer method of test. The essential features of the latter are thorough dispersion of the soil in water and determinations (a) of the density of the suspension thus formed by means of a sensitive hydrometer at successive intervals afterward and (b) of the maximum diameter of particles in suspension at each interval by means of Stokes's law. A special stirring apparatus¹ is used for dispersing the soil (see Fig. 5). Figures 6 and 7 show the dispersion cups and types of hydrom-

¹ The stirring device (Fig. 5) and hydrometer B (Fig. 7) were designed by Dr. G. J. Bouyoucos.

eters used. Graphical solutions of Stokes's law expedite the work.

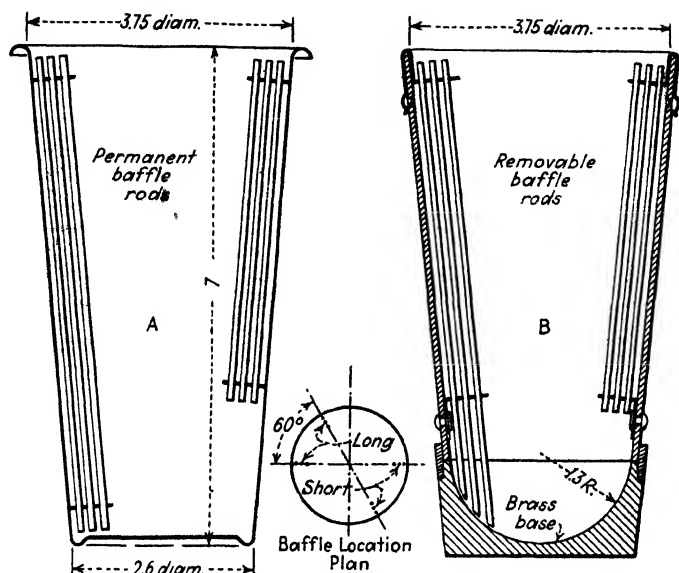


FIG. 6.—Dispersion cups.

The equation relating the reading of hydrometer *B* to the percentage of soil in suspension may be expressed as follows:

Let *H* = hydrometer reading.

W_s = weight of soil originally dispersed per liter of suspension.

x = weight of soil in suspension per liter of suspension.

ΔH = correction to hydrometer reading for variation, temperature from 67°F.

f = correction coefficient for variation, specific gravity from 2.65,

$$= \frac{2.6500 - 0.9984}{2.6500} \times \frac{G}{G - 0.9984},$$
 where *G* is the specific gravity of the soil.

P = percentage of originally dispersed soil remaining in suspension.

For the standard values of temperature and specific gravity (67°F. and 2.65),

$$x = H$$

For other values,

$$x = (H + \Delta H)f$$

The percentage of soil in suspension is given by the equation

$$P = \frac{(H + \Delta H)f}{W_s} \times 100 \quad (4)$$

This equation may be written

$$P = (H + \Delta H) \times \frac{f}{W_s} \times 100 \quad (4)$$

For the basic or standard conditions we have

$$\Delta H = 0, \quad f = 1, \quad W_s = 50$$

so that the basic relation is given by the equation

$$P = \frac{H}{50} \times 100 = 2H$$

In order that Stokes's law may serve to disclose the diameter of the soil particles, it is necessary to know the distance through which these particles fall in a given time. The distance L through which the soil particles are assumed to settle, in the determination of the grain size by means of

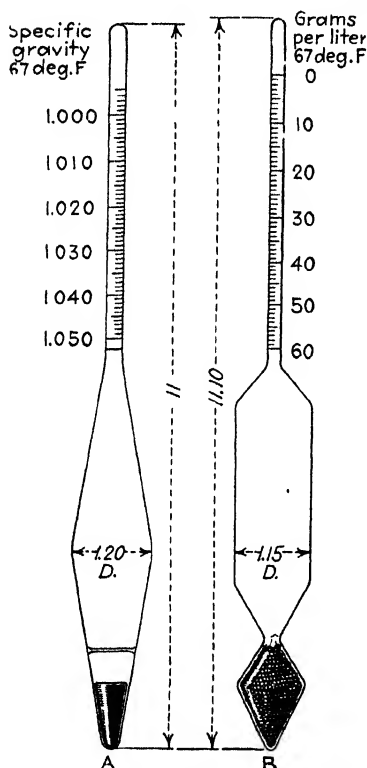


FIG. 7.—Hydrometers used in mechanical analysis of soils.

the hydrometer B , equals 0.42 of the total distance between the surface of the suspension and the elevation of the bottom of the hydrometer.

If a hydrometer like that designated as B (Fig. 7) is used, the essential features of the combined method are as follows:

The percentage of soil particles in suspension for each hydrometer reading, a percentage that depends upon the quantity of soil dispersed, the specific gravity of the soil, and the temperature of the suspending medium, is determined by use of Chart I (Fig. 8).

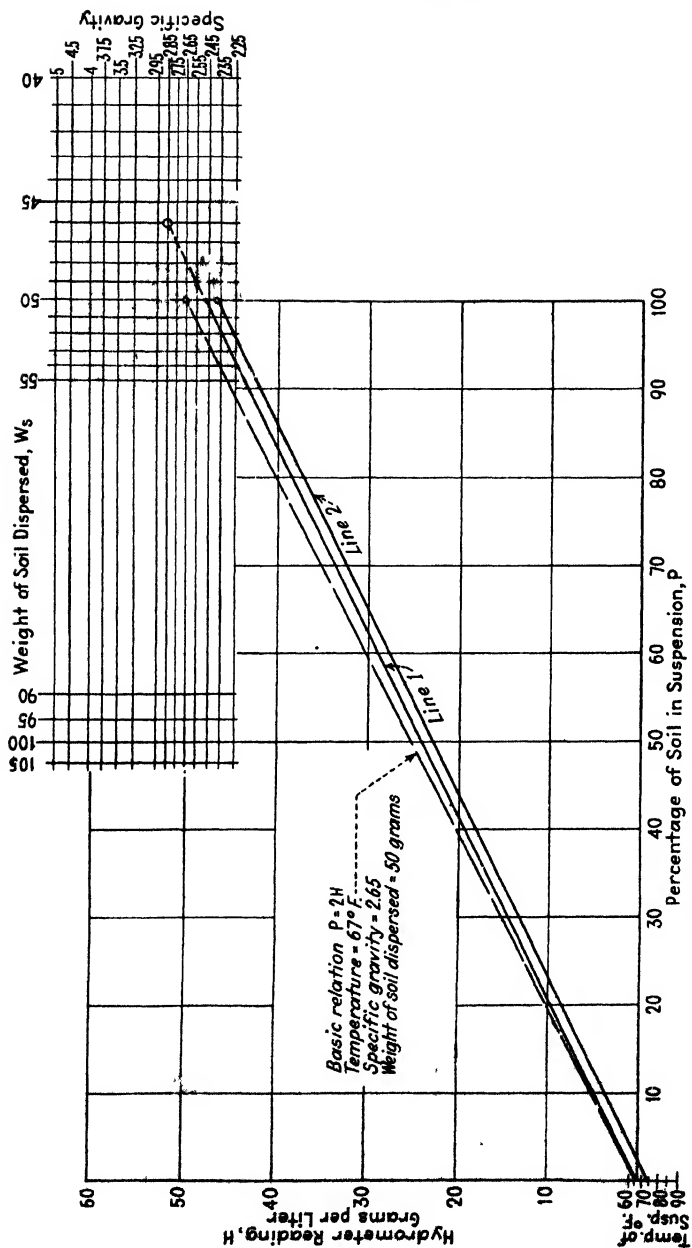


FIG. 8.—Chart used for converting hydrometer readings into percentages (Chart I).

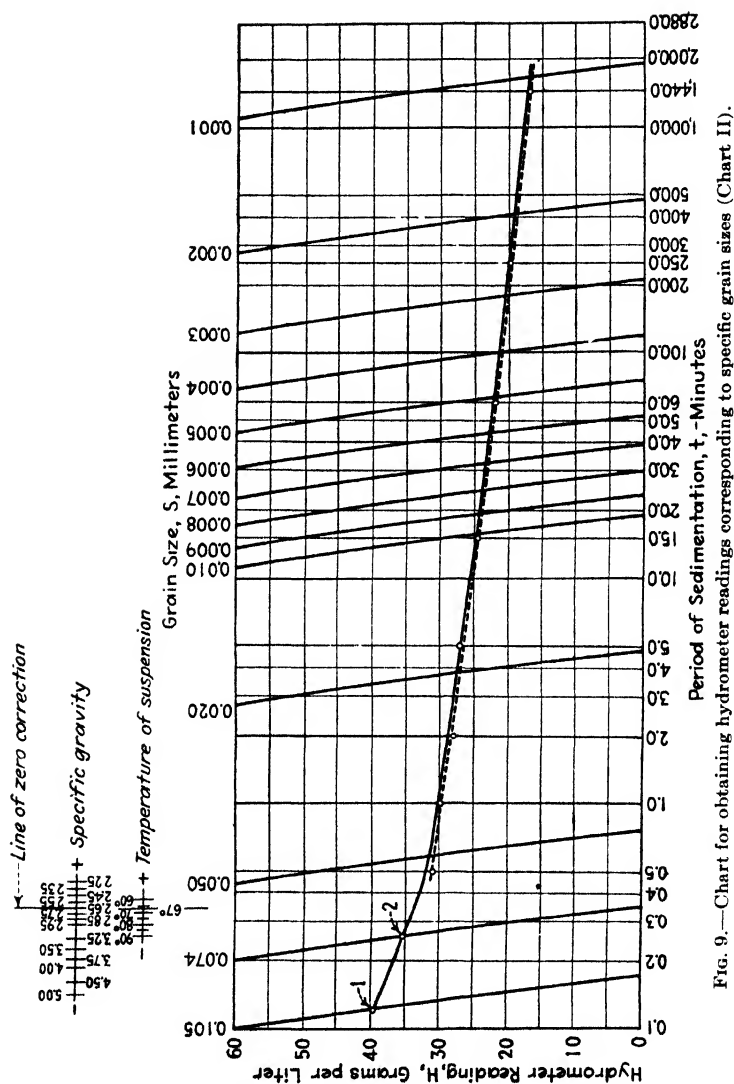


Fig. 9.—Chart for obtaining hydrometer readings corresponding to specific grain sizes (Chart II).

A line is drawn connecting the origin at *O* with the intersection of the abscissas representing the number of grams of soil dispersed and the ordinates indicating specific gravity of the soil particles. Parallel to this line another line is drawn passing through the given temperature indicated at the lower left-hand corner of the chart. This second line indicates the percentage of soil in suspension for any hydrometer reading.

The size of the soil particles in suspension at a given hydrometer reading is determined by use of Chart II (Fig. 9), which shows the relation between the hydrometer reading, the time of sedimentation, and the maximum grain size in suspension, according to Stokes's law, when the temperature of the suspension equals 67°F., and the specific gravity of the soil particles equals 2.65.

To determine the maximum grain size in suspension, a curve is plotted on a piece of transparent tracing paper placed over Chart II (Fig. 9), with hydrometer readings as ordinates and the periods of sedimentation in minutes as abscissas. This tracing paper is then moved to the right or left for distances corresponding to the corrections for temperature and specific gravity shown in the upper left-hand corner of the standard chart. The grain sizes corresponding to the determined hydrometer readings are then read through the tracing paper from the stand-

TABLE 6.—DATA USED IN PLOTTING ACCUMULATION CURVE SHOWN IN FIG. 10

Maximum Grain Diameter	Per Cent Finer
Sieve Analysis	
Inches	
1	100
$\frac{1}{2}$	87
Millimeters	
2.0	53
0.42	34
0.074	20
Hydrometer Analysis	
0.05	17
0.005	8
0.001	4

ard chart. The percentages of the soil remaining in suspension are determined from Chart I (Fig. 8). The hydrometer-analysis

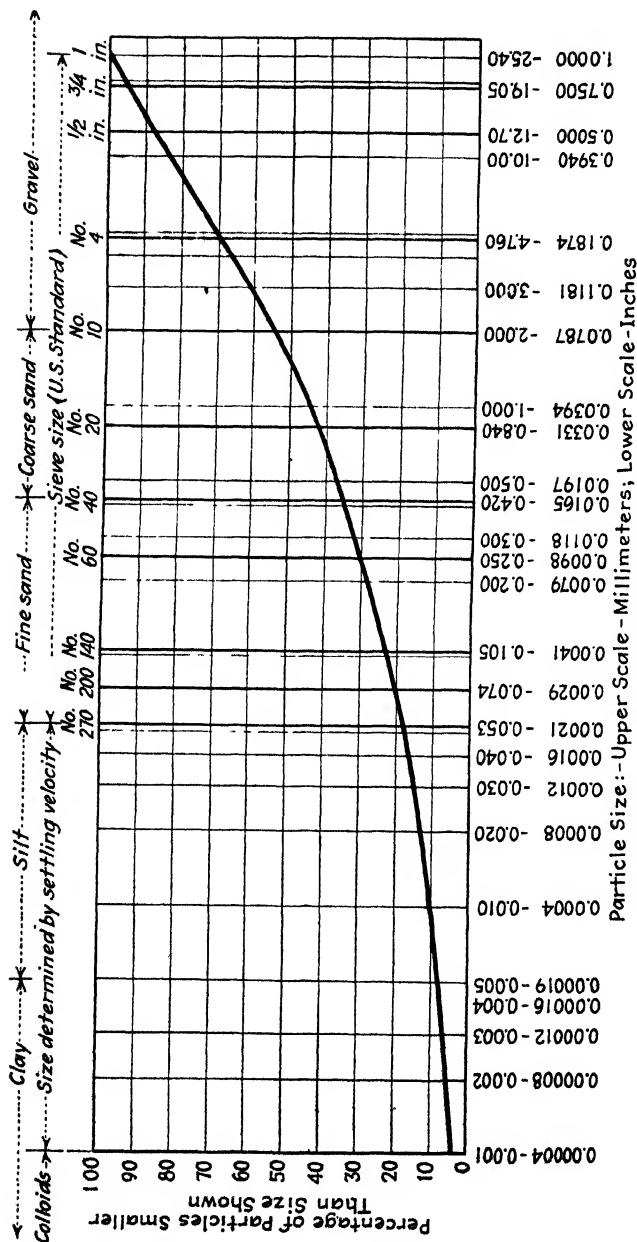


Fig. 10.—Particle-size accumulation curve.

results combined with the sieve-analysis results furnish values such as are shown in Table 6.

A convenient method of showing the distribution of particle sizes in a soil is illustrated by the chart (Fig. 10) in which grain diameters (Table 6) are plotted as abscissas on a logarithmic scale, with the corresponding total percentages as ordinates on an arithmetical scale. The curve connecting the points so plotted is termed the "particle-size accumulation curve."

Two terms used by Allen Hazen (57) for indicating particle-size distribution are (a) the effective size *es*, which is the maximum diameter of the smallest 10 per cent, by weight, of the soil particles; and (b) the uniformity coefficient *uc*, which is the quotient obtained by dividing the effective size into the maximum size of the smallest 60 per cent, by weight, of the soil particles.

Thus the abscissa of the point having an ordinate of 10 per cent on the distribution curve (Fig. 10) discloses the effective size—in this case 0.010 mm. The maximum size of the smallest 60 per cent of the soil particles, determined in the same manner, is 3.000 mm., giving a uniformity coefficient of $3.000/0.010 = 300$.

Effect of Electrolytes.—Suspensions of soil particles may be of three kinds: granular materials without colloids, colloidal clays in flocculated or coagulated state, and colloidal clays in the deflocculated or disperse state. The effect of these differences in the characters of the various materials is reflected first in the manner in which the suspended matter settles out of the suspensions and, second, by differences in the characters of the sediments thus formed (58).

When no colloids are present, particles of the coarser grained materials settle out of suspension independently and form stratified layers with the faster falling large particles on the bottom. Above this the successive layers are comprised of particles decreasing in size, with the smallest particles in the top layer as shown in Fig. 11A. In contrast with this, in suspensions of soils containing flocculated colloidal clay, the flocculated masses of colloids trap the larger particles and prevent independent settlement. As a result the clay particles are distributed throughout the highly porous sediments thus formed without stratification, as illustrated in Fig. 11B.

Deflocculents, which destroy the electrolytic bonds of colloidal flocs, cause the suspension to be in the disperse state and thus

effect the release of the clay particles to settle individually like bulky grains to form stratified sediments as shown in Fig. 11C. The colloids may then either remain in suspension or gradually form a gel on top of the sediment.

According to Searle (43), flocculation of clay may be effected by any of the following agents:

1. An acid having an easily adsorbed ion of the electric sign opposite to the clay.
2. An acid salt corresponding to 1.
3. A salt having the necessary easily adsorbed ions, such as alum.

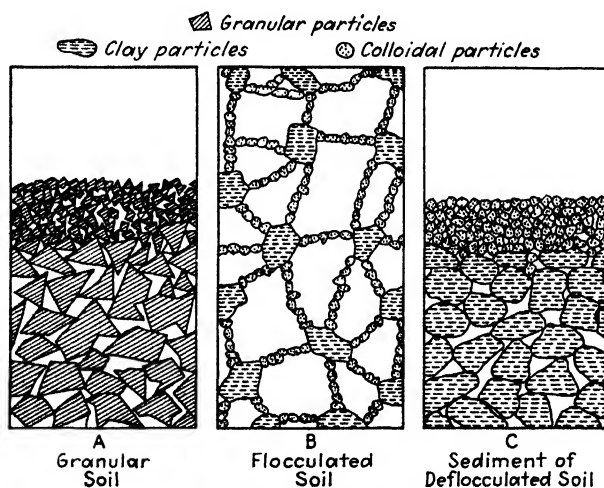


FIG. 11.—Types of soil sediments.

4. A colloid of sign opposite to the clay, *e.g.*, ferric hydroxide.
5. An electric current which decomposes a salt and liberates readily adsorbed positive ions which produce flocculation.

Among substances that lower the surface tension of the mass of clay and serve to cause deflocculation are

1. An electrolyte, such as a solution of an alkali (ammonia, soda, etc.).
2. A nonelectrolyte such as many organic liquids.
3. A deflocculated colloid, the particles of which carry the same electric charge as the clay.
4. A mixture of two or more deflocculating agents such as sodium carbonate and sodium silicate.

5. The removal of a flocculating constituent.

Soil particles carried in suspension by rivers, sometimes for great distances, flocculate suddenly and drop out of suspension at the mouth of the rivers where they flow into bodies of salt water.

Flocculation is a common natural phenomenon, being produced generally by the limes and magnesias while the sodas and potassium compounds and also certain physical agencies such as heat and agitation serve to deflocculate or disperse colloids in suspension.

Flocculation of the nerve colloids is the physical chemist's explanation of the anesthesia produced by ether, chloroform, and the like. Flocculation also accounts for the better drainage of farm lands after being limed. Milk is curdled by flocculation upon the addition of lime-carrying juices of fruits and vegetables. The value of sodium in bicarbonate form for preventing such curdling is well known to every housewife.

Flocculation Constants:—Figure 12 shows a flocculated layer to the left. It is uniform in appearance and has a moisture content of 130 per cent. The deflocculated layer on the right is stratified and has a moisture content of 59 per cent. Both were formed in 24 hr. from suspensions of equal amounts of soil material, the one on the left being prepared with distilled water alone, and the one on the right with distilled water and a small amount of a chemical deflocculent.

The moisture content of a deflocculated sediment obtained in this manner is defined as the "flocculation limit." The corresponding ratio of pore volume to that of solids is defined as the "flocculation factor." Conversion of flocculation factors into flocculation limits, and vice versa, is accomplished by the substitution of numerical values in formula (2) (page 33).

The flocculation limit of the muck soil is 269 per cent. The corresponding flocculation factor is 7.29. Generally, the floccu-



FIG. 12.—Flocculated and deflocculated soil sediments.

lation factors of typical soil materials are as follows: coarse sands, 0.6 or less; fine sands, 0.6 to 0.8; ground rock powders, 0.8 to 1.2; average silty soils, 0.8 to 1.6; colloidal clays, 1.6 to 3.5; mucks, peats, and highly micaceous soils, 5 to 8.

The pore-solids ratio (flocculation factor) of spheres of equal size has been shown (59) to equal approximately 0.35 in the densest possible state and 0.90 in the loosest arrangement possible with the spheres still in contact. Consequently, the more

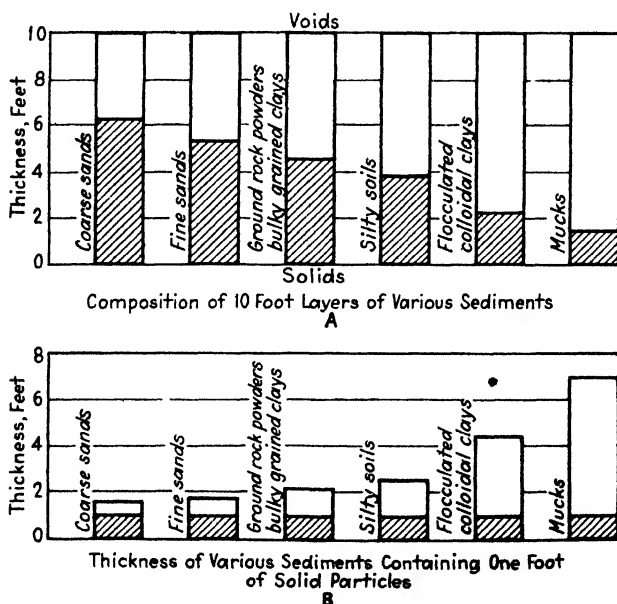


FIG. 13.—Effect of flocculation on the thickness of soil deposits.

the flocculation factor of a sediment exceeds 0.90 the more is indicated the presence of colloidal flocs which tend to separate the larger grains.

The effect of flocculation upon the thickness of deposits, illustrated in Fig. 13, is of special interest in connection with the construction of hydraulic fills. Figure 13A indicates the lower densities which cause the muck and clay deposits to be so much softer than fills of silt and sand.

A deposit of muck 7 ft. thick (Fig. 13B) has the same amount of solids as a deposit of silt 2.5 ft. thick and a layer of sand 1.5 ft. thick. Therefore, if the price of muck per cubic yard were but

5 cts., the value of a cubic yard of silt or sand, based on the amount of solids furnished, would be, respectively, 14 and 23 cts.

Problems

1. (a) Define density and specific gravity.
(b) What is the difference between absolute density and bulk density?
(c) What is the difference between absolute and bulk specific gravity?
2. Define (a) porosity, (b) voids ratio, and (c) moisture content.
3. A sample of completely saturated soil received in the laboratory is found to have a moisture content w of 40 per cent, and the specific gravity G of the soil particles is 2.65.
(a) Compute the porosity and voids ratio of the soil sample.
(b) What is the absolute density in grams per cubic centimeter and in pounds per cubic foot?
(c) What is the bulk specific gravity?
(d) What is the bulk density in grams per cubic centimeter?
(e) What is the bulk density in pounds per cubic foot?
4. The sample of soil referred to in Prob. 3 is completely immersed in water.
(a) Compute the absolute density in c.g.s. and f.p.s. units.
(b) Compute the bulk specific gravity.
(c) Compute the bulk density in c.g.s. and f.p.s. units.
5. (a) On what basis are soils classified geologically?
(b) Name and define the classes.
(c) State the soil-transporting agencies and name a soil material deposited by each agency.
6. Define soil texture and name the principal textural classes.
7. What is meant by (a) gumbo, (b) caliche, (c) peat soil, (d) muck soil, (e) alkaline soil, (f) acid soil, and (g) blow sand?
8. (a) What are pastes?
(b) What is the difference between a plastic material and an elastic material?
(c) What is the difference between plastic soils and friable soils?
(d) What is a slurry?
9. (a) Define colloidal state.
(b) Name the chief colloidal systems.
(c) What is a "colloidal sol?"
(d) How is a "gel" formed?
10. Upon what does the speed of settlement of soil particles in suspension depend?
11. How long would it take for a particle of soil 0.005 mm. in diameter to settle from the surface to the bottom of a lake 10 ft. deep when the specific gravity of soil G equals 2.60, the coefficient of viscosity n of the water equals 0.0102, and the density of the water G_1 equals 1.000?
12. What are the essential features of the combined sieve and hydrometer method of determining the grain size of soils?
13. Define (a) effective size and (b) uniformity coefficient.

14. Discuss the settlement from suspension of granular soil particles without colloids, colloidal clays in flocculated state, and colloidal clays in the deflocculated state.

15. Mention two methods of making clays (a) flocculate and (b) deflocculate.

16. (a) Define flocculation limit.

(b) Define flocculation factor.

(c) A soil has a flocculation limit of 74 and a specific gravity of 2.68. What is the flocculation factor?

CHAPTER III

NATURAL SOIL FORMATIONS

The suitability of any part of the earth's crust for plant growth and for use as subgrades and as foundations for bridges, buildings, and the like depends upon the soil constituents present, their arrangement in a particular soil layer, and the relation of this layer to those above and below it.

The Soil Body.—A vertical section of the soil deposit or body in the natural state reveals a definite construction, or build, consisting of distinct layers, known as horizons. The soils in these layers are specific in their morphological characteristics irrespective of the geographical position or the underlying geologic formation, provided they are located in identical climatic zones.

The parent material can be either native rock upon which the soil body has been formed or some material that may have been a part of a former soil body, as in the case of the so-called transported soils, or it may even be some geologic formation like peat, clay, marl, or sand. Soil material is therefore not to be identified with the soil as a natural body. A soil body ceases to be one as soon as its virgin make-up has been disturbed; it continues to be soil material from which a soil might form again in the course of time, primarily under the influence of the active agents, the climate and the biosphere.

Various parent materials will give rise to one and the same type of soil, provided all other conditions for the activities of the other soil-forming materials are alike. The soil type known as chernozem may be found on such variable parent material as loess, glacial deposits, marine and lake sands and clays, limestone, sandstone, and shale. Also, different soil types will develop on one parent material, provided the conditions for the activities of the other soil-forming materials are not alike. Thus granites in Georgia yield the typical yellow-red soil, whereas the same granites in Southern California form a different kind of soil.

The Soil Profile.—Dr. C. F. Marbut (60), the prominent American soil investigator, of the Bureau of Chemistry and Soils,

U. S. Department of Agriculture, divided the soils of the United States into two large groups: (a) *pedalfers* and (b) *pedocals*. The *pedalfers* are soils that tend to accumulate iron and aluminum and have no lime carbonate horizon accumulation, even if the soils have limestone as parent material. The accumulation of lime

carbonate and other salts is a characteristic feature of the *pedocals*.

The prefix "ped-" comes from the word "pedology" which is the term applied to the study of soils from the standpoint of the agriculturist; "-alfer" is apparently an abbreviation for aluminum and ferrum (iron); and the "-cal" in *pedocal* comes from the word "calcium."

A vertical section of the weathered soil layers, having different characteristics though uniformly weathered within themselves, is known as the soil profile (see Figs. 14, 15, and 16). Soil profiles are identified with respect to textural types and series, a series being a group of soil types which have



FIG. 14.—Profile of Iredell silt loam showing (1) light-textured upper soil layer, (2) heavy, plastic, sticky, waxy clay, and (3) decomposed rock.

profiles uniform in all respects except the texture of the surface layer. The different series are designated by such names as Chesterfield, Putnam, Berks, Susquehanna, etc. These names indicate the localities where the particular soils were first studied or where they attained considerable development.

The terms used to describe soils in the profile may be summarized as follows (see Appendix IV, page 408):

Texture: sands, clay loams, etc., according to the textural classification described above.

Color: black, red, brown, yellow, drab, gray, mottled.

Structure: buckshot, granular, columnar, cloddy, crumb, adobe, dense, massive, laminated, nut, mealy, structureless.

Consistency: brittle, hard, compact, tough, tenacious, sticky, plastic, cheesy, friable, mellow, loose.

Compactness: clay pan, hardpan, degree of compaction.

Cementation: cementing material and degree of cementation.

Chemical composition and properties: significant chemical constituents and properties are iron, calcium, concretions, alkalis, sesquioxides, exchangeable bases, and pH values

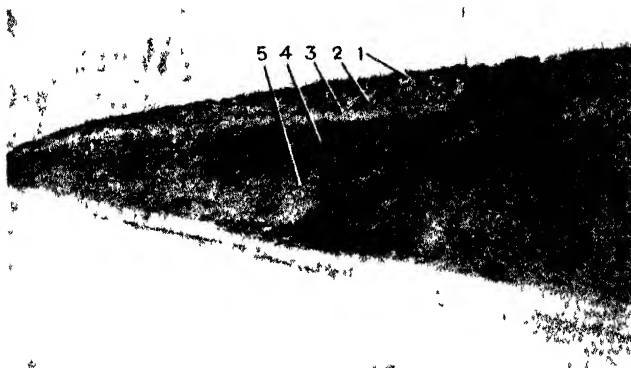


FIG. 15.—Profile of Leonardtown silt loam showing (1) grayish brown granular silt loam, (2) brown granular silty clay loam, (3) gray compact sandy hardpan, (4) friable clay, and (5) compact clay

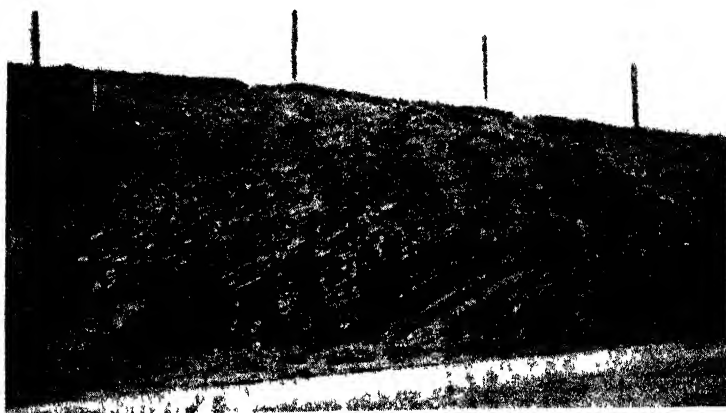


FIG. 16.—Profile showing stratifications in shale.

In well-drained soils of the humid regions the profile consists of a light-textured upper layer designated by the agronomist as

the *A* horizon underlain by a heavier textured layer known as the *B* horizon. A still lower third layer of unweathered or incompletely weathered material is known as the *C* horizon, or parent material. Each horizon or layer may be subdivided, as *A*₁, *A*₂, *B*₁, *B*₂, etc.

Extensive soil surveys carried on by the U. S. Bureau of Chemistry and Soils (formerly the Bureau of Soils) during the past twenty-five years have resulted in the publication of soil maps and descriptive matter on soils covering more than half the total area of arable land in the United States.

Highway and civil engineers are interested only in those properties upon which the serviceability of pavements or other structures depends, properties that are often of minor significance in an agricultural sense. In other cases, the scale adopted in the publication of reports of agricultural soil surveys is too small to show all the details essential from an engineering standpoint. For this reason profiles mapped for engineering purposes differ in several respects from those described above. They are drawn to larger scale, which permits much greater detail; they generally include a considerably greater depth, with layers numbered instead of being designated as horizons; and they are not necessarily identified by series and types but by group designation of the classification based upon engineering properties.

Methods Used in Determining the Soil Profile.—The soil profile is determined by examining the soil in its natural field condition. Vertical sections are examined at frequent intervals and classified into layers identified according to definite changes in texture, structure, consistency, compactness, chemical composition, etc. Particular attention should be paid to the location of water-bearing strata, a knowledge of which is essential to the proper design and location of drains, and of layers of highly plastic material which may be productive of sliding.

For highways and airports the depth to which the examinations are carried should be sufficient to penetrate the more or less nonuniform layers of soil material. This work is best accomplished by examining excavations such as road cuts or test pits, but the method commonly used is to examine samples obtained at successive depths by use of a soil auger. To expedite the work of making a soil survey, such borings can be made by unskilled assistants, who need only to lay out, adjacent to each

test hole, samples obtained at regular intervals for inspection by the soils engineer.

The examination of foundation soils for buildings, bridges, and other heavy structures and for embankments such as highway fills and earth dams must be carried to the depth of solid rock or other unyielding material. The procedure for such examinations is essentially the same as that used in highway and airport soil surveys, except that special equipment and technique for making the borings and for obtaining samples are required.

The detailed mapping of the soil profile within a given section is accomplished by plotting the limits of the various layers at

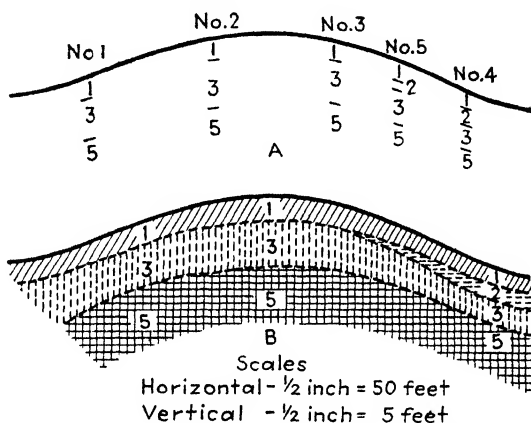


FIG. 17.—Example of soil profile mapping

each test hole, as shown in Fig. 17A. The consecutive layers at the first test hole are indicated by alternate numbers (1, 3, 5, etc.) so as to allow the inclusion of any additional layer which may enter the profile within the section mapped. The occurrence of additional layers at subsequent test holes will require the examination of additional test holes between those first made, in order to determine the horizontal limits of such additional layers. The map of the profile is completed by connecting the points marking the limits of the layers at each test hole, as shown in Fig. 17B.

In addition to the field examination, samples representative of each layer are obtained for laboratory test. These samples are taken either in disturbed or undisturbed state according to the purpose for which information is required. A sufficient

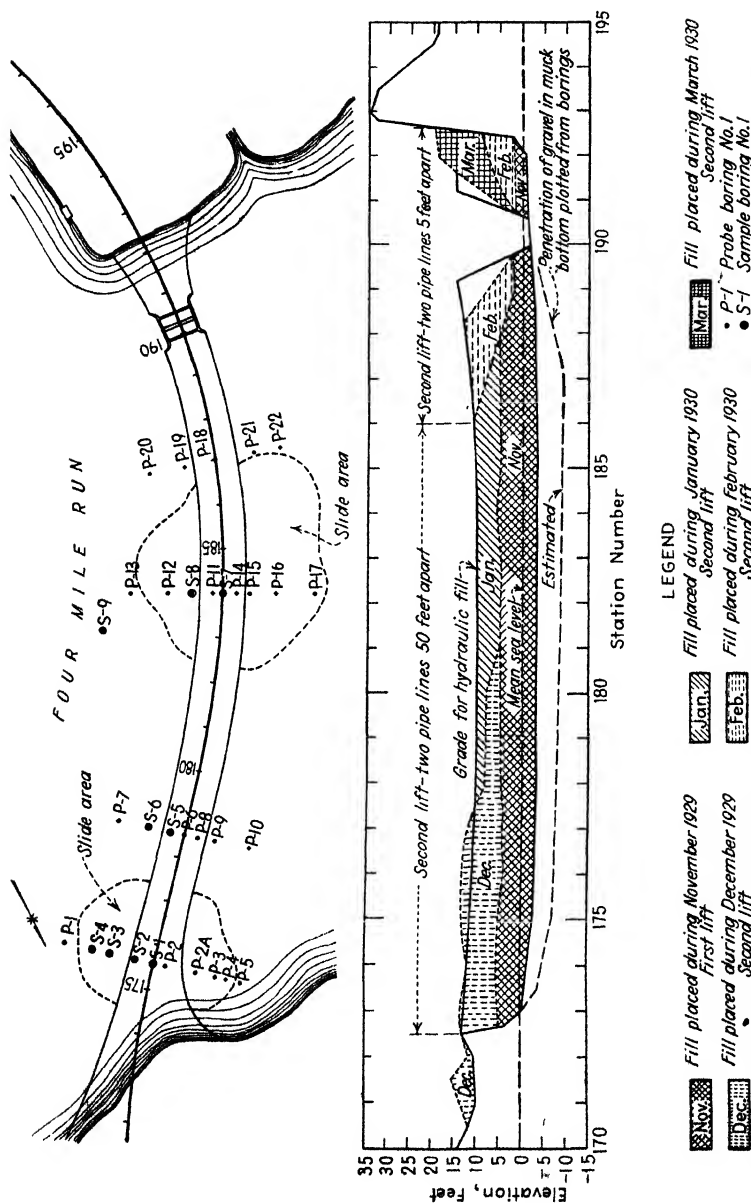


Fig. 18.—Location of borings and progress in constructing hydraulic fill at Four Mile Run, Mount Vernon Memorial Highway.

number of samples are taken to determine the range in test results for what appears to be the same layer.

Obtaining Undisturbed Samples of Soil.—The methods used in the study of the hydraulic fill settlement at Four Mile Run on the Mount Vernon Memorial Highway are typical of those used in the exploration of foundation soils (61). The location of the borings to determine the cross section of the fill and to obtain samples of the compressed muck layer is shown in Fig. 18.

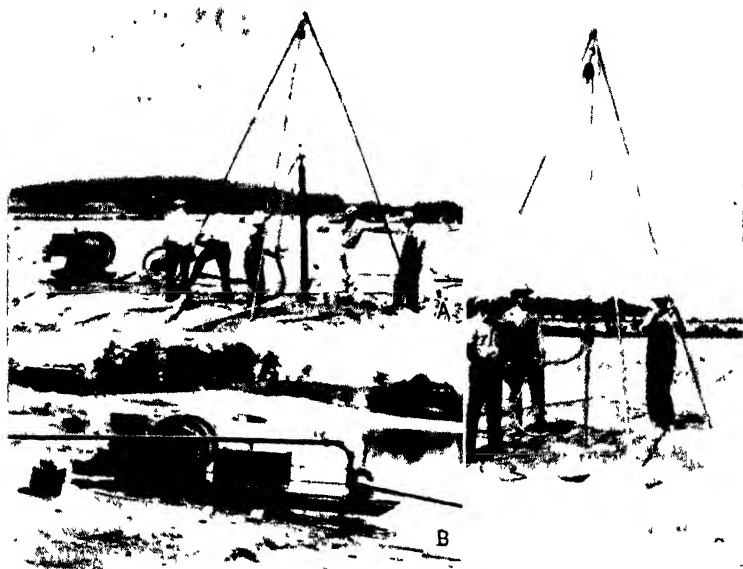


FIG. 19.—*A*, apparatus for taking undisturbed cores; *B* and *C*, apparatus for probe drilling.

They were of two kinds: “probe” borings, designated as *P*-1, *P*-2, etc.; and “sample” borings, designated as *S*-1, *S*-2, etc.

The probe borings were made by driving a 2-in. pipe through the fill material and soft undersoil until firm foundation was reached. As the driving proceeded, the material penetrated was washed up through the 2-in. pipe by water pumped through a 1-in. drill pipe. These borings indicated only the elevation of the bottom of the fill material and the bottom of the muck. A setup of this boring equipment is illustrated in Fig. 19.

In making the sample borings a 4-in. pipe was used for the casing, and a 2-in. pipe for drilling and washing in the same

manner as for the probe borings. As the drilling proceeded, cores of river-bottom material were obtained in an undisturbed state from different elevations in the muck layer.

The equipment used in driving and washing the casing is illustrated in Fig. 19.

The device for taking cores consisted of a section of 3-in. galvanized-steel pipe about 16 in. long, sharpened at one end and attached by a special coupling to a 1-in. pipe at the other. Inside the coupling was a check valve, specially designed, as

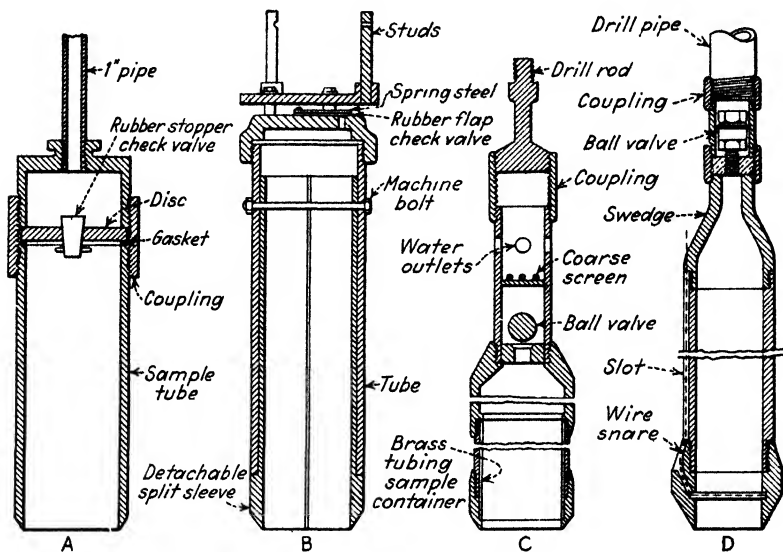


FIG. 20.—Devices for obtaining undisturbed soil cores.

illustrated in Fig. 20A, to prevent the loss of the core when the pipe was withdrawn.

Three other types of sampling devices are shown in Fig. 20 (62). Type B was developed by Glennon Gilboy and used by the Committee on Foundations of the American Society of Civil Engineers in their work at the Massachusetts Institute of Technology. Moran and Proctor used type C for sampling the foundation soil at the site of the Bay Bridge across San Francisco Bay. Type D, with the unique feature of a piano-wire snare for cutting the sample at the bottom, was designed by Arthur Casagrande and Spencer Buchanan.

In devices *A* and *C* part of the apparatus is used as the container for sending the undisturbed soil to the laboratory. In samplers *A* and *D* vacuum may be applied to the sample from the surface of the ground. A limited number of tests on cores obtained in these two samplers indicate that the snare is advantageous in sampling highly compacted deposits.

For sampling soft materials in disturbed state the projectile sampler designed by Murray and Flood (63) embodies advantageous features:

The undisturbed samples at the Four Mile Run fill were taken in the following manner:

The 4-in. casing was first thoroughly cleaned of all the material penetrated, after which the coring device was lowered through the casing and pushed down into the river-bottom material



FIG. 21.—Sampling device and sealed cores.

slowly and without twisting. The air and water entrapped by the soil entering the device escaped through the check valve. When the resistance to further penetration indicated that the device was filled with soil, it was turned through two revolutions and then pulled up slowly. The tube containing the soil core was next uncoupled, sealed with paraffin, and labeled. Figure 21 shows two cores sealed with paraffin and the sampling device for obtaining these cores.

At depths other than those at which the cores were taken, samples of the material washed up through the casing were obtained and placed in glass jars of 1 qt. capacity.

The elevations from which both cores and samples were obtained in the sample borings are shown in Fig. 22, which shows also the character of the material penetrated as identified in the field.

Method of Sampling Described.—The procedure used by the Bureau of Public Roads for obtaining undisturbed samples of soil with the device shown in Fig. 20D is described as follows:

A casing having an internal diameter of 6 in. is forced down to the desired depth. The material penetrated by the casing

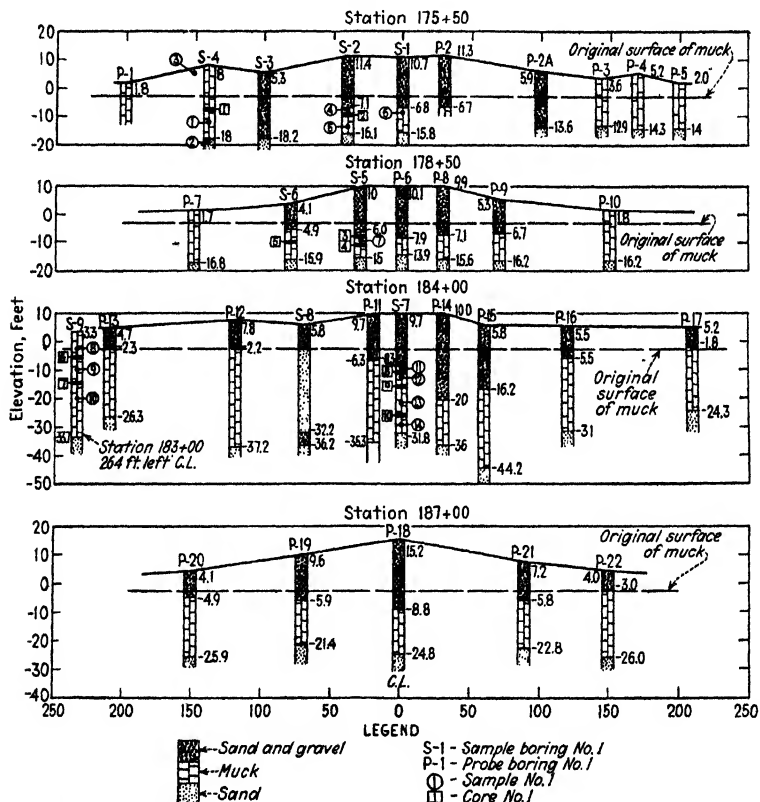


FIG. 22.—Locations of samples and character of materials, Four Mile Run fill, Mount Vernon Memorial Highway.

is removed by means of a water jet. The water and solid matter remaining in the casing is bailed out, and the hole sponged dry.

The sampling device (Fig. 20D) is coupled to the necessary length of drill pipe and lowered to a firm seat on the bottom of the hole, care being taken to keep the snare cable slack so

as to prevent drawing the snare. It is driven, without twisting, by means of a 350-lb. drop hammer until the top of the sample just enters the swedge.

The driving connections are removed, and the drill pipe is connected to the vacuum side of a vacuum-pressure pump. About 15 in. of vacuum is applied, and after the tools in the hole are lifted $\frac{1}{4}$ in., the cable is pulled sufficiently to shear off the sample at the bottom with the piano-wire snare.

The drill pipe is disconnected from the vacuum-pressure pump, and the tools withdrawn from the hole. The sampling device is placed in a horizontal position on a table top, the check valve removed, and the swedge connected to the pressure side of the vacuum-pressure pump. With the necessary air pressure the sample is forced out of the sampling device on to a clean piece of waxed paper in such a manner as to avoid subjecting the sample to bending as it leaves the device. The sample is wrapped in waxed paper and carried on the table top to a table frame located under a shelter which prevents the sun and wind from drying the sample during the preparation for shipment.

Upon arrival in the shelter the best appearing 12 in. of the sample is sawed out, immediately wrapped in waxed paper, and placed on end on a prepared paraffin disk about 1 in. thick. A galvanized-metal cylinder, 14 in. high and about 6 in. in diameter, is slipped over the sample and paraffin disk so that it rests on the table top leaving an annular space of about $\frac{1}{2}$ in. around the sample and about 1 in. between the top of sample and top of cylinder. The cylinder is then completely filled with molten paraffin and allowed to remain undisturbed until the paraffin has hardened.

Other methods of subsurface exploration are the earth-resistivity and seismic methods (64) used in the determination of the presence and location of consolidated rock, the location and extent of gravel deposits and quarry material, and the preclassification of materials to be excavated in grading operations.

Resistivity Method of Exploration.¹—The procedure most commonly used in the resistivity method of subsurface exploration is to introduce an electric current of known strength into the

¹ Digest of a report by E. R. Shepard, research engineer, Bureau of Public Roads.

earth through suitably placed electrodes and by means of intermediate electrodes to measure the average resistivity of a volume of earth comparable in dimensions to the electrode spacing. By progressively increasing the electrode spacing and repeating the measurements, a curve is established which shows the relation of resistivity to depth.

Calculated values of resistivity are plotted as ordinates, and corresponding values of electrode spacing as abscissas. An upward trend of the curve indicates increasing resistivity with depth and suggests the presence of rock, gravel, or some other high-resistance material, while a flat or descending curve is

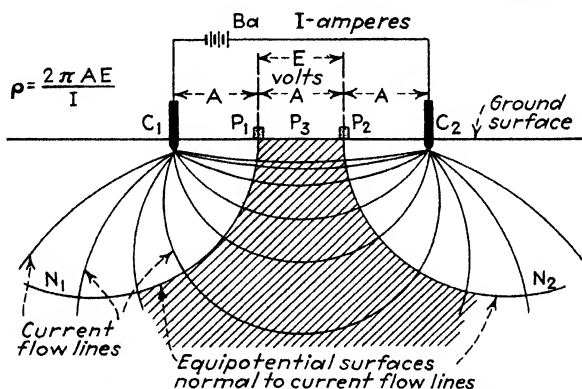


FIG. 23.—Diagram illustrating theory of earth-resistivity measurements.

indicative of soil or clay. An abrupt change in curvature indicates a change in material at a depth approximately equal to the electrode spacing at which the change occurs.

The principle involved in making earth-resistivity measurements is illustrated in Fig. 23. An electric current I is caused to flow through the earth between two electrodes C_1 and C_2 . Measurement is then made of the potential drop E between two intermediate electrodes P_1 and P_2 placed in line with C_1 and C_2 and dividing the distance C_1C_2 into three equal parts A termed the electrode spacing.

In 1915 Wenner (65) showed that with this four electrode arrangement the specific resistivity of a homogeneous medium given by the equation

$$\rho = \frac{2\pi AE}{I} \quad (5)$$

where A is expressed in centimeters. The specific resistivity p is expressed in ohm-centimeters and denotes the resistance in ohms between parallel faces of a centimeter cube of the material. Where A is measured in feet we may write:

$$p = 191A \frac{E}{I} \quad (6)$$

If the medium is not homogeneous, the indicated resistivity p represents an average specific resistivity of the material bounded by the equipotential surfaces P_1N_1 and P_2N_2 but its value is determined largely by the properties of the material near the surface and only slightly by the material at a relatively great depth. The experience of numerous investigators is rather consistent in showing that for practical depth determinations the materials within a depth equal to the electrode spacing A largely determine the value of p and that materials at a depth greater than A have a negligible influence on the value of p .

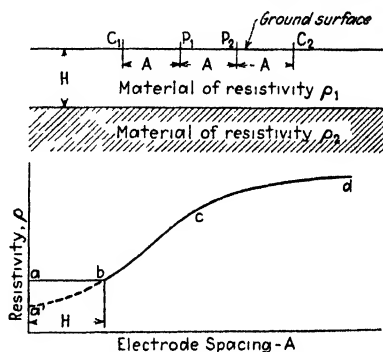


Fig. 24 — Typical resistivity curve.

Let us assume in Fig. 24 that a homogeneous soil of resistivity p_1 exists to a depth H and that below the horizon H a material of greater resistivity p_2 exists. If, under this condition, earth-resistivity measurements are made with different values of electrode spacing A , as indicated in Fig. 23, and a curve is plotted showing the relation of p to A , as in Fig. 24, we find that for values of A less than H the curve ab is approximately flat but that when A begins to exceed H the influence of the deeper material begins to affect the value p , and the curve assumes an upward trend as at b . Rather abrupt changes of this kind are often obtained in practice and afford a means of roughly determining the depth to a change in material. This is a simple empirical rule but one for which there is no theoretical basis.

Tagg (66) and others have shown that from theoretical considerations the curve in Fig. 24 should assume some such

form as a/bcd with no sharp breaks or inflections. In practice, however, and especially on shallow work, rather sharp inflections are frequently obtained where materials of different resistivities are concerned, and these inflections afford a simple and satisfactory means of ascertaining the approximate depth where the change in material occurs.

Perhaps a more common occurrence is a U-shaped curve in which the electrode spacing at the low point is taken as the depth to a change in material. The descending branch of the curve is characteristic of many soils in which a decreasing resistivity occurs with increase in depth. Under such conditions an underlying stratum of rock or gravel is indicated quite accurately for the low point of the curve.

Resistivity measurements carried out by the Bureau of Public Roads have been confined largely to experimental studies of known formations for the purpose of developing suitable apparatus and methods of test and to determine the conditions under which this method of exploration may be of value in planning highway and bridge construction.

Perhaps the most significant features brought out by these studies are the wide range in resistivities of both soil and rock encountered within a comparatively limited area and the lack of uniformity and regularity in the upper layers of the earth's crust. Because of these conditions the interpretation of resistivity measurements is often indefinite or uncertain, and frequent correlations with known conditions should be made wherever possible.

Although precise classification cannot be expected, it should be possible to identify two and possibly three kinds of material, such as earth, broken or loose rock, and consolidated rock. The contractor is primarily interested in knowing if he can handle the material with his shovel or whether he will need to resort to blasting. The resistivity test can give him the answer in the great majority of cases.

Seismic Method.—The seismic method of exploration is based on differences in the speed of sound waves through soil and rock. Nonrigid matter such as sand, clay, or gravel transmits sound waves at velocities of 1,000 to 6,000 ft. per second, while rigid materials like rock or crystalline matter transmit such waves at 16,000 to 20,000 ft. per second. Thus is provided

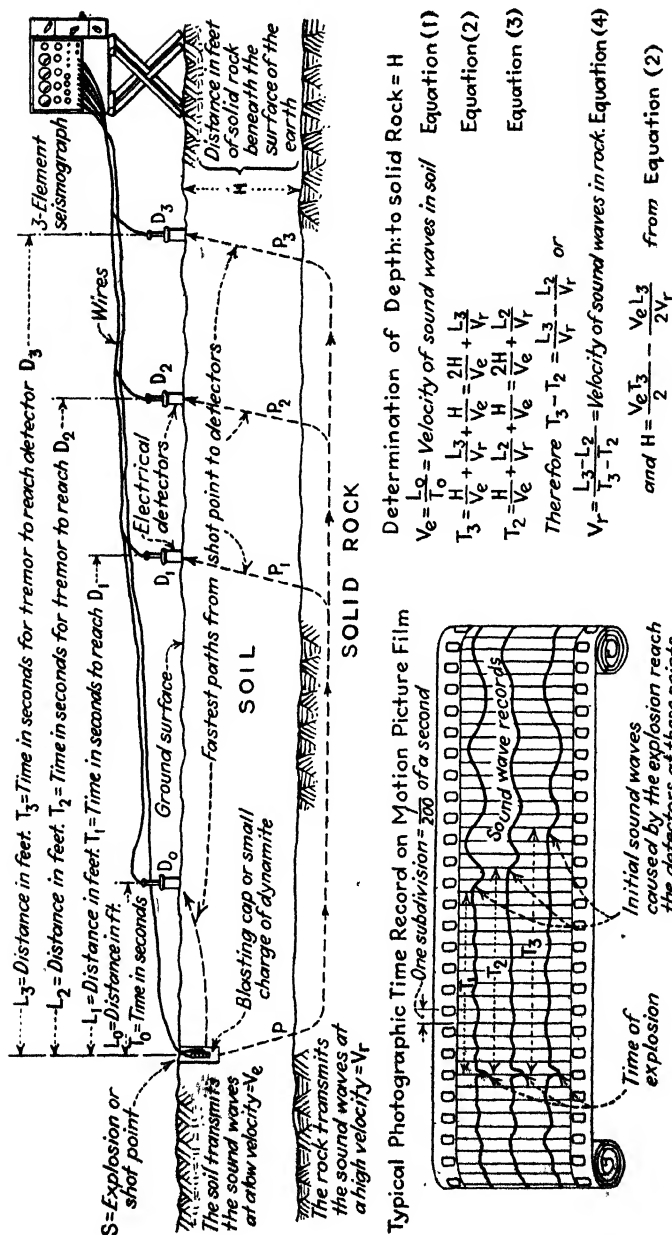


Fig. 25.—Diagram showing fundamental principles of seismographic method.

a means of measuring directly the property of subsurface material with which the engineer is concerned, *viz.*, rigidity, which can be measured only indirectly by the resistivity method. The seismic method is, therefore, more definite in the identification of solid rock than the resistivity method but cannot so readily be used to distinguish between clay and coarse sand or gravel.

Two methods of seismic exploration that are in common use are known, respectively, as reflection shooting and refraction shooting. In reflection shooting the interpretations are based on waves that are reflected directly from different strata under investigation, while in refraction shooting it is the refracted waves that are studied. As reflection shooting is not well

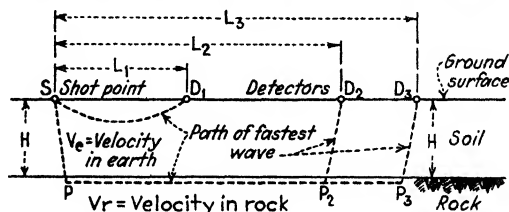


FIG. 26.—Wave propagation in seismic exploration.

adapted to shallow explorations, no further space will be given to description of that technique.

The principles involved in refraction shooting are illustrated in Figs. 25 and 26. A blasting cap or small charge of dynamite is exploded at or under the surface of the ground at some point *S*, which becomes the center of a wave disturbance which moves outward on a spherical or near-spherical front in all directions. Detectors *D*₁, *D*₂, and *D*₃, placed on a line passing through *S*, pick up this disturbance successively and carry the impulses to three galvanometers which record them as light traces on a moving photographic film.

A small wire is wound around the blasting cap and arranged so that when it is broken by the explosion an initial kick or impulse is given to the three galvanometers, and the time of the explosion is thereby indicated on all three traces. As the wave front moves outward from *S*, its time of arrival at each of the three detectors is indicated on the moving film by disturbances in the traces. By counting the number of time units between the initial kick and the arrival of the wave at each detector, it is possible to calculate the average velocity of wave propagation

from shot point to each detector. From data of this kind it is usually possible to determine with a fair degree of accuracy the depth to the first stratum of rigid or consolidated rock and to obtain other information of value regarding the character of the subsurface material.

With reference to Fig. 26, the path of the first wave to reach any detector will depend upon the ratio of the shooting distance L to the depth to rock H and also upon the relative velocities of wave propagation in the two media. If the shooting distance is relatively short, such as L_1 , the path of the first disturbance to reach D_1 will be directly through the soil as indicated. The velocity of the wave through the earth can then be calculated from the equation

$$V_e = \frac{L_1}{T_1} \quad (7)$$

As the shooting distance is increased, a point such as D_2 will be reached where the wave first to arrive will no longer be that going directly through the top soil but that taking the path of the refracted wave SPP_2D_2 . Subsequent disturbances will reach detector D_2 by various paths; but as only the first arrival can be definitely identified on the film record, it is necessary to base interpretations solely on so-called "first events." Likewise, the first wave to reach D_3 will be by the path SPP_3D_3 . By measuring the time difference $T_3 - T_2$ and the distance $L_3 - L_2$ we are able to calculate the velocity of wave propagation through the rock from the equation

$$V_r = \frac{L_3 - L_2}{T_3 - T_2} \quad (8)$$

This relation is obvious from the fact that the time difference $T_3 - T_2$ is taken up by the travel of the wave through the rock from P_2 to P_3 , since that part of the path SPP_2 is common to both circuits.

We are now able to write the equation by which H , the depth to rock, can be calculated from known quantities. This is based on the fact that T_3 , the total time of travel of the wave from S to D_3 , is the sum of the combined times of wave travel over the path SPP_3D_3 . Although refraction of the wave at the point P is supposed to take place at the angle of total internal

refraction, we may assume with negligible error under practical conditions that this angle is 90 deg. and that the distance PP_3 is equal to L_3 . With this assumption we may write:

$$T_3 = \frac{H}{V_e} + \frac{L_3}{V_r} + \frac{H}{V_e} = \frac{2H}{V_e} + \frac{L_3}{V_r} \quad (9)$$

or

$$H = \frac{V_e T_3}{2} - \frac{V_e L_3}{2V_r} \quad (10)$$

When the distance L_3 is of such value that the direct wave through the earth reaches detector D_3 at the same time as the refracted wave, it is said to be the critical distance, and we may write:

$$T_3 = \frac{L_3}{V_e} = \frac{2H}{V_e} + \frac{L_3}{V_r}$$

or

$$H = \frac{L_3}{2} \left(1 - \frac{V_e}{V_r} \right)$$

and

$$L_3 = \frac{2H}{1 - \frac{V_e}{V_r}} \quad (11)$$

For best results, when determining depths to rock, L should not be greatly in excess of the critical distance. As a fairly close approximation we may say that for values of V_e and V_r , usually found, $H = 0.45L$, or $L = 2.2H$, where L is the critical shooting distance.

Tests made at quarry sites and at other locations where solid rock is found within a few feet of the ground surface have indicated that the depth of such overburdens can be determined quickly, definitely, and accurately by the seismic method. For such work a single No. 8 blasting cap gives ample energy, thus reducing the cost of explosives to a negligible amount.

Problems

1. Discuss the formation of soil types with respect to their parent materials and climate.
2. Name the groups into which the soils of the United States are divided.
3. What are the distinguishing characteristics of these groups?
4. Define "soil profile."

5. What is meant by (a) soil type and (b) soil series?
6. What characteristics of the soil are used in describing the different layers of a soil profile?
7. How is the soil profile determined?
8. Describe briefly the steps involved in making borings for the exploration of foundation soils and obtaining samples in an undisturbed state.
9. Describe briefly the procedure for making subsurface explorations by (a) the resistivity method and (b) the seismic method.

PART II
CHARACTERISTICS OF SOIL

CHAPTER IV

COLLOIDAL SURFACE PHENOMENA

Standardized methods of determining the properties of rock and soil **aggregates** and of bituminous and Portland-cement binders individually have long been used by the engineer.

The development of research on soil stabilization, however, has demonstrated the need for additional methods of examination to throw more light on the adhesion between **aggregate** and binder.

The required information concerns particles of soil, sand, crushed rock, gravel, slag, etc., coated with films of air, water, soluble chemicals or binders not soluble in water; the relative adhesion between solids and films; and the manner in which this adhesion is influenced by the chemical composition of the solid particles and the kinds of metallic ions on their surfaces.

To obtain a rational explanation of how water alone or water supplemented by calcium chloride, sodium chloride, or other chemicals effects soil stabilization, the engineer has had to include in his studies the electrochemical phenomena of adsorption and base exchange long recognized in agriculture and ceramics.

Adsorption, according to Bancroft (67), is the phenomenon that causes all solids to adsorb or condense on their surfaces any gases or vapors with which they are in contact. When adsorption involves reactions that are essentially chemical or ionic in character, it is termed base exchange. The role of adsorption in soil performance involves complex colloidal phenomena. Fortunately for those interested in soil studies the effects of these phenomena can be observed in many commonplace happenings and well-known industrial and manufacturing processes. Bancroft (67) calls attention to a number of familiar occurrences and operations involving adsorption, which facilitate the describing of soil characteristics.

Adsorption Illustrated.—Film performance depends upon electrical attractions which may be illustrated by a toy horseshoe magnet. A piece of steel is attracted strongly enough by a

magnet to cling to it and be lifted against the force of gravity. A piece of copper, in contrast, is not so attracted by the magnet. If the two metals could be combined in varying proportions, the magnet would be expected to attract the resulting alloys in varying degrees covering the whole range from none for the copper to a maximum for the steel.

Similarly, all solids attract with varying intensities any gases, vapors, or even solids with which they are in contact (67). In the air, solids adsorb films of air; and in saturated soils, they are covered with minute films of moisture. In the kitchen, molds are wetted before cornstarch or gelatin is put in; pans are buttered before fudge is poured in; and griddles are greased before flapjacks are cooked on them. Cast-aluminum griddles do not need to be greased, because an adsorbed air film keeps the cakes from sticking. For a liquid to wet a solid in air, the liquid must displace the air film, and, for this to occur, the solid must attract the liquid more strongly than it attracts the air.

Dried soil clods retain moisture enough to bind the particles into hard and, in some instances, enormously strong masses. On the other hand, air is often adsorbed on soil solids so strongly after a period of drought that drops of rain will roll along on dust without wetting it.

Water wets glass; mercury does not. Therefore, to be strictly logical, we must conclude that a column of mercury in a glass tube does not touch the glass at any point and that it stands alone surrounded by a film of either mercury vapor or something that is not glass.

A special case of nonwetting which depends upon volatility was observed by Dewar, 1878 (67), when pieces of solid carbon dioxide were dropped into water. The dioxide did not become covered with ice, although its temperature was considerably below that of the arctic regions. It was in reality coated with a layer of gas constantly renewed, which prevented the solid from actually coming in contact with the water.

A drop of water, as from an oar of a boat or paddle of a canoe, may fall on a sheet of water, become submerged, and then emerge with the upper portion of the drop apparently unwetted by the liquid with which it has been covered. .

We can oil the meshes of a fine-mesh sieve and carry water in it because the oil prevents the water from wetting the wires.

Consequently, surface tension prevents the water from passing through the holes.

All campers are familiar with the unpleasant fact that touching canvas with one finger will cause a tent to leak in the rain. It matters not whether the touching is from the inside or the outside. It either contracts the air space and drives out the air so that the water enters when the finger is removed, or it expands the air space at that point, and water enters.

The reverse case is to be seen in the ordinary water wings. They are made of a fine-meshed fabric which is first thoroughly wetted so that a film of water is formed in the pores. The strength of this water film is so great relatively that one can blow up the water wings without the air's escaping, and the water wings will then support a grown person without difficulty.

Adsorption Is Selective.—Globules of fat in cold broth are attracted to and adhere very strongly to any metal spoon. The removal of the fat from the spoon, by a soap or a soda which has greater attraction for the metal than the fat has, demonstrates the basic principle of all cleansing operations.

Water displaces kerosene in contact with quartz; kerosene, water in contact with copper; and linseed oil, water in contact with white lead. Consequently wet cloths remove kerosene from glass and a covering of vaseline or heavy oil protects metal surfaces from rusting. If a mixture of carbon and silica gel is shaken with benzene and water, the benzene will go to the carbon, and the silica to the water.

Only those paints which are attracted more strongly than moisture films by steel structures are likely to prevent the formation of a layer of rust under the paint coat. This is true no matter how dry the surface when the paint is applied. In oil sands, possibly as much as half the oil exists as coatings on the sand grains. After all of the free oil has been pumped from the wells, more oil is recovered by displacing the film oil from the sand grains with water or, better still, with sodium carbonate solution.

The fact that diamonds are wetted more readily by grease than by water enables them to be rapidly separated from their ore. The disintegrated blue earth containing the diamonds is mixed with water and passed over shaking tables smeared with grease. The diamonds adhere to the grease, and the other constituents flow off as tailings.

The removal of air is not necessarily sufficient to make a liquid wet a solid such as glass. It has been found that the nature of the glass has a distinct bearing in the case of mercury. The more alkaline the glass the harder it is to make mercury wet it. Fairly good wetting can generally be obtained with quartz glass, while it is only occasionally possible to wet Pyrex or soda-lime glass with mercury.

Clay adsorbs different liquids to different extents; *i.e.*, its power of adsorption is selective, and under suitable conditions one liquid will displace another in contact with the clay. For the same reason a greasy dish is readily cleaned by rubbing it with wet clay, and grease may be removed from wool and cloth by rubbing it with dry clay.

The power that plastic clays possess of retaining their plasticity when mixed with sand or other nonplastic material is due to the adsorption of one solid by another. The clay is not distributed uniformly through the pores or interstices of the coarser particles, but most of it forms a coating on the nonplastic material, and many of the pores remain unoccupied even though there is more than sufficient clay to fill them.

Film Characteristics.—Materials in film phase have properties differing widely from those of the same materials in bulk phase. In general, the films exist under enormous pressures with corresponding high densities and are unbelievably thin. They are influenced by change in the electrical properties of both the solid and the adsorbed material and have great strength.

Two plane surfaces placed closely together will adsorb appreciably more of a gas than the same surfaces spaced farther apart. With the same solid and the same gas the amount of adsorption is greater with increased pressure of the gas and less with increased temperature.

Hardy and Miss Nottage in 1928 found that a pressure of 14 lb. per square foot reduced the air films between two plane surfaces to a thickness of about 160 millionths of an inch, or 1,500 molecules (67).

Pettijohn (68) found about 5 millionths of an inch for the maximum thickness of water films on beads made from one type of glass and 10 millionths of an inch for beads made of another type of glass. With river sand the estimated thickness varies from 20 millionths of an inch with sand retained on the No. 10

sieve to 5 millionths of an inch with sand retained on the No. 60 sieve. Meyer found the stationary film on copper to be 150 millionths to 225 millionths of an inch thick when water flowed rapidly and about three times as thick when water flowed more slowly over the copper.

Some conception of the enormous density of adhesive films is furnished by the thought that the transition of a pure liquid to its own vapor is not abrupt but that over a narrow range all the densities intermediate between those of vapor and those of the liquid actually occur. Williams (69) has suggested that the first layer of an adsorbed gas vapor may be under a pressure of as much as 10,000 atmospheres, graded down from the corresponding density to that of the liquid in bulk in the outermost layer. The transition film for carbon dioxide is said to be about 6 hundred-millionths of an inch.

A nitrogen cylinder filled with dry cocoanut charcoal will discharge far more nitrogen on equal reduction of pressure than a similar cylinder containing no charcoal. It is believed that the sudden outbursts of gas in coal mines involve gaseous adsorption; the gas is held in the coal in a state available for discharge when the pressure is released. These outbursts are characterized by an almost instantaneous discharge into the mine of thousands, and in some cases of millions, of cubic feet of firedamp.

It has been found that electrical resistance increases with increasing adsorption and that mercury wets platinum only when an electric current is flowing. At other times there is evidently an air film. A vertical jet of water breaks into drops which scatter because an enclosing film of adsorbed air causes the drops to rebound when they strike one another. A feebly electrified body brought close to the jet causes the air films to be removed to such an extent that the drops coalesce and form a coherent stream.

It is possible to press together two soap bubbles, which are merely hollow drops, with considerable force without causing them to coalesce unless they are electrified slightly, when they coalesce readily without bursting. Dr. F. E. Hance has observed that samples of soil which resist slaking indefinitely when immersed in distilled water may disintegrate almost instantaneously when immersed in drinking water from the faucet containing electrolytes in solution. •

The very fine vapor films have an adhesive power so great that they cannot be removed from glass by heating at a temperature up to 500°C . or from soils by forces of from 9,000 to 15,000 atmospheres. Keen (47) showed that samples of sand grains with binder-clay colloids can be nineteen times as strong in compression as similar samples of sand grains with an equal proportion of Portland-cement binder.

In the frosted-glass industry, gelatin or glue coatings drying on rather thick glass exert a tenacity high enough to tear away the surface of the glass itself, chipping it into fernlike patterns. A brittle glue will give a pattern different from that of a tough glue, and the addition of salts also modifies the patterns.

Wetting Power.—Various means have been devised for measuring the wetting power of a liquid or the attractive force that a liquid exerts on a solid. As early as 1896 different forms of apparatus were described (see Bancroft) in which water evaporates through a porous tube or a porous cup of plaster into dry air. As the water evaporates off, mercury is drawn up into the tube and, if bubbles of air do not

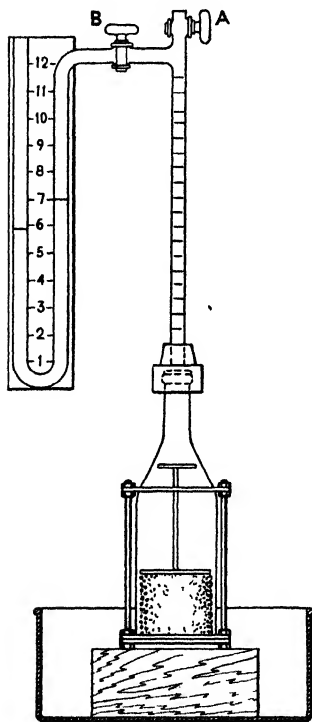


FIG. 27.—Drainage indicator.

form, can reach incredible heights, limited by the tensile strength of the liquid.

The drainage indicator described before the Highway Research Board (70) in 1932 utilized this principle, as does one modified form shown in Fig. 27. The latter consists of a glass container with perforated-metal bottom, manometer, graduated glass tube, and valves arranged as shown. Water introduced through the valve *A* passes down the graduated tube into the glass container to the soil sample held between a perforated plate above and the perforated bottom of the glass container below.

With the valve *A* closed and the valve *B* open, the device serves to disclose the resistance that the moisture exerts to prevent being expelled from the soil pores. This is a measure of the adhesive attraction of moisture for the soil particle which is responsible for the rise of capillary moisture in the pores of soils.

Base Exchange.—It was stated in Chap. II that all materials in nature seem to have electrical properties like those responsible for the Brownian movement of the solid in colloidal solutions (43). The charge of the soil particle is determined by the inner ion shell on the surface of the particle, which is covered by another shell of opposite charge (71) (see Fig. 28).

A colloid that holds hydrogen adsorbed on its surface is termed a hydrogen-ionized or hydrogen colloid. One with calcium adsorbed on its surface is termed a calcium-ionized or

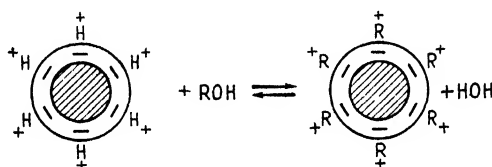


FIG 28 - Ion shells of soil particles

calcium colloid. If a substance like hydrated lime, $Ca(OH)_2$ is leached through soil containing hydrogen colloids, the calcium replaces the hydrogen to form calcium colloids, and the hydrogen thus released combines with the $(OH)_2$ to form water.

According to Searle (43), the theory of base exchange had its beginning in 1850. At this time, and also in 1852, J. T. Way (72) reported to the Royal Agricultural Society that soils are able to remove ammonia from its solution, a fact appearing still more extraordinary inasmuch as there is no ordinary combination of ammonia in a state of insolubility in the soil. The removal of ammonia was due to its replacing lime in the clay, the latter passing into solution. Later, Eichorn (1858) (73) showed that natural double silicates, known as zeolites and used largely as water softeners, have this power of exchanging one base for another.

Weigner (1912) (74) showed definitely that base exchange of silica gel has all the characteristics of an exchange adsorption process. The amount of cations adsorbed is mathematically

related to the concentrations of the surrounding solutions at equilibrium.

Turner (75) working with tropical soils was able to evaluate the exchange capacities both of the organic matter and of the clay. Bradfield (1923) (76) furnished information on the types of acids that occur in soils. Arrhenius (1926) (77) found that the adsorption of dyes by clay is related to their molecular weights, materials having the higher molecular weights being adsorbed to the greater extent.

Work at the University of Missouri by Dr. Hans Winterkorn and Dr. L. D. Baver (78) has done much to substantiate and supplement the findings of the earlier investigators on the effects of the chemical characteristics of the soil upon its physical properties.

Replaceable Ions.—Of the two parts of hydrogen and one of oxygen (H_2O) that comprise water, one part of the hydrogen (H) may be considered as positive acid-producing ions; the other part of the hydrogen in combination with the oxygen (OH) as the negative alkaline-producing hydroxyl.

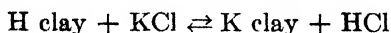
Distilled water free of dissolved carbon dioxide is neutral; *i.e.*, the hydrogen and the hydroxyl ions are in equilibrium. Acids or acid-producing materials dissolved or dispersed in distilled water cause the hydrogen or acid ions to dominate in the resulting solution or suspension. The hydrogen colloids then result. Bases or base-producing materials dissolved or dispersed in distilled water cause the hydroxyl or alkaline ions to predominate in the resulting solution or suspension.

The relative acidity or alkalinity is indicated by the pH value which may be defined as the logarithm of the reciprocal of the grams of ionized hydrogen per liter of solution or suspension. Thus the total ionized hydrogen in grams per liter equals 10^{-pH} . The pH of distilled water equals 7.0. The lower the pH's below 7.0 the greater is the acidity of the liquid. The higher the pH's above 7.0 the greater is the alkalinity of the fluid.

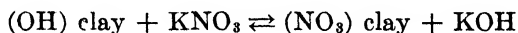
The determining factor as to which ion will be adsorbed is the charge on the particle. If the particle is silica, which is negatively charged, the cation (positive acid-producing ion) will be adsorbed. Conversely, if the particle is composed mainly of iron or aluminum compounds which carry positive surface charges, the anion (negative base-producing ion) will be adsorbed.

Base exchange is controlled in a qualitative manner by the kind of ions and in a quantitative way by the concentration of the ions. Base exchange with univalent and bivalent ions is reversible, but with hydrogen and trivalent ions some of the hydrogen and trivalent ions are fixed in the soil colloid and cannot be replaced.

The leaching or percolation of the neutral salt solution through silica soil saturated with adsorbed hydrogen is productive of base exchange whether the soil be a small sample in a funnel in the laboratory or a layer of material in natural state in the soil profile in the field. Typical reactions are as follows:



and



Two influencing factors in base exchange are "buffer action" and "mass action."

The effect of buffers is to furnish resistance to change in the hydrogen-ion concentration. Their effect is illustrated by the following statement by Russell (79):

Each 10 per cent of the amount of base needed for neutralization changes the pH value only by 0.2, yet the same amount of base added to pure water might have changed the pH by four or five units. This slowing down of the change of pH is called buffer action, and solutions that show it are said to be highly "buffered."

Information on the effect of mass action is furnished by Searle (43). He states:

What is sometimes termed the law of mass action states that, broadly, a reaction will proceed further or more easily if the masses of the reacting substances are large than if they are small. Thus, a typical example of mass action is shown by the fact that, whilst many silicates are unaffected by powerful acids in the laboratory, they are readily decomposed by feeble acids (such as a solution of carbon dioxide in water) when exposed in large quantities, as in the "weathering" of natural rocks.

Exchange Capacity of Soils.—The more acid the soil the greater is its base-exchange capacity, consequently its corrosive properties, whether they refer to the destruction of metal pipes, etc., or to admixtures of chemicals, etc., used for the purpose of

furnishing stability. The corrosive properties are illustrated by Fig. 29, which shows the relationship between the depth of corrosion of metal pipes and the pH of the soils in which the pipes were buried (80).

However, in order that base exchange may approach completion even in acid soils, the metallic salt which is introduced must have opportunity to percolate through the soil mixture in solution and carry away the displaced hydrogen. If the displaced hydrogen ions are not carried away by percolation, they inhibit the occurrence of any base exchange in excess of the

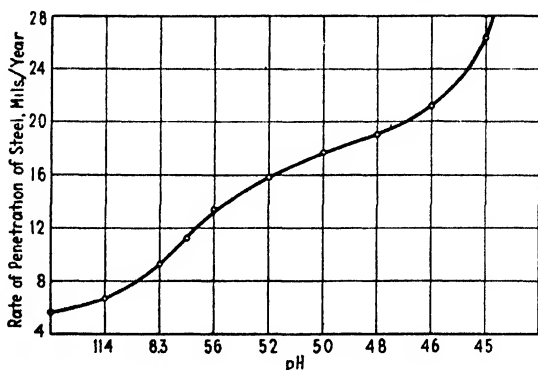


FIG. 29.—Relation between hydrogen-ion concentration and corrosion of steel in synthetic soil.

limited amount required to create a condition of equilibrium, after which there is no further exchange. Now, water percolating through the treated soil dissolves the salt, washes away displaced hydrogen ions, and tends to destroy equilibrium with the result that further base exchange occurs.

The determination of the hydrogen-ion concentration of a suspension of the soil by means of pH measurements may serve to furnish some idea of the amount of ionized hydrogen.

Let $\text{pH} = \text{acid index} = \text{logarithm of reciprocal of the hydrogen-ion concentration of the suspension.}$

x = grams of soil in suspension.

W_w = cubic centimeters of distilled water which is the suspending medium.

$[\text{H}^+] = 10^{-\text{pH}} = \text{grams of total ionized hydrogen per liter of solution or suspension.}$

Then the total ionized hydrogen TH in percentage of the weight of soil may be determined from the expression

$$TH = \frac{W_w \times [H^+] \times 100}{1,000x} = \frac{W_w \times 100}{1,000x} \times 10^{-pH} \quad (12)$$

If, then, the pH of a suspension of 3 grams of soil dispersed in 10 cc. of distilled water equals 4, the percentage of ionized hydrogen is

$$TH = \frac{10 \times 100}{1,000 \times 3} \times 10^{-4} = \frac{1}{3} \times 10^{-4} = \frac{1}{30,000} \text{ per cent}$$

On this basis the theoretical quantity of metallic salt N required for complete replacement of the ionized hydrogen, in percentage of the weight of the soil, equals the product obtained by multiplying TH by the equivalent weight of the salt. The equivalent weight is defined as the quotient obtained by dividing the molecular weight of the salt by the valence of the exchangeable metallic ion.

For example, the molecular weight of calcium carbonate, CaCO_3 , is about $40 + 12 + 48 = 100$. The valence of the exchangeable calcium is 2. The equivalent weight of calcium carbonate then equals $100/2$, or 50. Therefore, the amount of calcium carbonate, in percentage of the weight of the soil, required to furnish the calcium that will completely replace hydrogen present to the amount of $1/30,000$ of the weight of the soil equals $50/30,000 = 1/600$ per cent.

As a matter of fact, complete saturation of the soil with the calcium ions requires the presence of a neutralizing salt in amounts considerably in excess of the theoretical amounts computed in the manner just shown.

The total amount of replaceable hydrogen for the several clays investigated by Winterkorn and Baver is shown by the exchange capacity (Table 7). These data show the grams of exchangeable hydrogen per 1,000 grams of clay. Thus in the Putnam clay the replaceable hydrogen was found to be 0.65 gram per kilogram of soil.

The percentage of this total exchangeable hydrogen which may occur in the absence of percolation is shown in Table 8. Thus the percentage of completion for the hydrogen-Putnam clay by calcium is 26.9 per cent. In other words, without percola-

tion, calcium ions may replace 0.175 gram of hydrogen per 1,000 grams of soil colloid. In the absence of leaching, but 26.9 per cent of the total available hydrogen would be exchangeable.

TABLE 7.—PROPERTIES OF H COLLOIDS OF DIFFERENT CLAYS

Type of H colloid	SiO ₂ /R ₂ O ₃ ratio	Colloidal property			
		Ex-change capacity, gram per kilogram	Swelling, cubic centimeter per gram	Hygroscopicity, per cent	Heat of wetting, calories per gram
Bentonite.....	5.0	0.85	2.20	21	
Lufkin clay.....	3.8	0.82	1.18	20.1	15
Wabash clay.....	3.2	0.78	0.94	14
Putnam clay.....	3.2	0.65	0.81	18.1	14
Susquehanna clay.....	2.3	0.47	0.57	15.5	12
Cecil clay.....	1.3	0.13	0.05	6.1	6

TABLE 8.—PERCENTAGE OF TOTAL EXCHANGE CAPACITY EFFECTIVE IN ABSENCE OF PERCOLATION

Colloidal system	Nature of cation					
	Li	Na	K	Ca	Ba	H
NH ₄ Putnam clay.....	29.9	35.3	51.3	63.6	71.7	84.9
H Putnam clay.....	6.6	6.2	14.5	26.9	23.8	
H bentonite.....	18.0	14.6	18.4			

Adsorbed Ions Affect Performance of Soils.—Research in base exchange by Joseph and Oakley (81) disclosed that generally the plasticity and shrinkage were greatest in soils saturated with lithium ions. As the kind of adsorbed ions was changed successively to sodium, magnesium, calcium, potassium, ammonium, and hydrogen ions, the plasticity was gradually reduced.

The shrinkage was gradually reduced as the kind of adsorbed ions was changed successively to sodium, magnesium, calcium, hydrogen, ammonium, and potassium. The moisture equivalents were highest for the sodium-ionized clay and were reduced

gradually as the kind of adsorbed ions was changed successively to lithium, magnesium, hydrogen, calcium, and potassium.

Joseph (82) gives an example of a sodium clay having a moisture equivalent double that of the calcium clay. The values were natural soil, 69 per cent by weight; sodium-ionized clay, 125 per cent; and calcium-ionized clay, 63 per cent.

The effect of adsorbed ions upon the swell and slaking values of clays as found by Winterkorn (78) is shown in Tables 7 and 9.

According to Russell (79), the normal clay of fertile soils is a calcium clay, but the calcium can easily be replaced by other bases or by hydrogen; a new clay then arises. Two of these clays—the acid and the sodium—occur somewhat frequently in nature.

TABLE 9.—INFLUENCE OF ADSORBED IONS ON SLAKING TIME
In minutes

Type of soil	SiO ₂ /R ₂ O ₃	Natural soil	Time of slaking for soil having ions of							
			Magnesium	Hydrogen	Potassium	Iron	Aluminum	Barium	Sodium	Calcium
Cecil.....	1.3	147	209	122	121	93	65	61	24	24
Hagerstown.....	...	15	49	26	35	15	20	32	341	23
Putnam.....	3.2	38	31	...	24	26	420	26

Calcium, sodium, and hydrogen clays are most common. Additional information on these types of clay is furnished by Taylor (83) as follows:

Since the clay fraction of a soil shows marked flocculation phenomena, it indicates that the clay particles are charged. The present conception is that the charge on the particle is negative, due to the anions of the clay, and surrounding them are the positively charged cations, the hydrogen, calcium, magnesium, potassium, and sodium. It is thus possible to have a series of clay types according to the cations present. The most important clay types are hydrogen clay, characteristic of an acid soil; calcium clay, the normal constituent of agricultural land and of silt carried and deposited in fresh water; and the sodium clay, which is the characteristic clay in salt and alkaline land. From the geological

point of view, calcium clay and sodium clay are the most important, as they are characteristic of fresh water and marine conditions, respectively. When a silt containing calcium clay is carried in suspension and deposited in water containing sodium chloride, the sediment will contain sodium clay as a characteristic constituent.

Other methods by which sodium clay may be formed are (a) the submergence of strata in sea water; and (b) by the capillary rise of sodium chloride solutions from a subsoil water table containing this salt in solution. These two latter modes of formation are characteristic of deltaic deposits. It will be seen, therefore, that sodium clay is a characteristic constituent of sediments formed under marine, estuarine, or deltaic conditions and that calcium clay is only associated with fresh-water sediments.

According to Taylor (83), the sodium clay remains flocculated in the presence of sodium chloride and is stable in the presence of an excess of that salt. On replacement of the salt solution with pure water, however, the rate of percolation decreases, the percolate becomes alkaline, and finally the sodium clay becomes impermeable.

Russell (79) states: "Sodium clay easily hydrolyzes. . . . It is very sticky, impervious to air, and dries into hard, large lumps which break down in water to a paste. It is difficult to work."

According to Searle (43), the more highly dispersed the hydrogen or H clay the more acid it can be at the optimum silica content.

According to Taylor (83), the properties of calcium clay differ considerably from those of sodium clay. The hydrolysis of calcium clay is a much slower process, and the calcium clay is flocculated in the presence of fresh water. It is neutral, pH value from 6.5 to 8.5, according to the amount of carbonic acid or calcium bicarbonate present, and holds more water in the air-dry condition than under wet conditions. It is less sticky, more permeable, and therefore drains more easily.¹

¹ The foregoing discussion concerns sodium- and calcium-ionized clays found in nature or produced in the laboratory. How far it applies to chemically treated and thoroughly compacted road soil mixtures is not known. The ionized clays can come into existence only when water has completely leached out and replaced the chemical salts with which the clays may have been treated. Under no conditions, then, are the properties of the sodium and calcium clays representative of the properties of the same clays that contain the calcium or sodium salts.

Problems

1. Define differential adsorption.
2. (a) How do materials in film phase differ from those of the same material in bulk phase?
 - (b) What is the maximum thickness of a water film on glass?
 - (c) Does mercury wet platinum?
3. How can wetting power be measured?
4. What controls the electrical charge on the soil particle?
5. What do pH determinations measure?
6. In base exchange reactions what factors control the base exchange?
7. If a common salt solution is leached through a hydrogen clay, what type of clay results?
8. (a) What is meant by buffer action?
 - (b) Define mass action.
9. If an iron pipe were placed in a soil having a pH of 8.1, should you expect more or less corrosion than if the soil had a pH of 3.8? Why?
10. Given a soil with a pH of 2.6 calculate the amount of sodium carbonate, Na_2CO_3 , required for complete displacement of the ionized hydrogen.
11. A clay has an exchange capacity of 0.82 gram per 100 grams of soil. Calculate the amount of replaceable hydrogen per square yard of soil road surface 4 in. thick compacted to a density of 135 lb. per square foot and containing 10 per cent of the clay.
12. Name the order of the adsorbed ions that cause a reduction in plasticity of clays.
13. What ion showed the least swelling in a series of ionized clays?
14. What ion is best adapted to form a soil having a crumb structure and good drainage?
15. What is the relationship between adsorbed ion and plasticity? Shrinkage?
16. What types of clays, with respect to adsorbed ion, occur most frequently in nature? What are the physical characteristics of each?

CHAPTER V

THE CHARACTER OF SOIL MOISTURE

Moisture films and water particles present in the soil mass, like the solids comprising the soil mass, vary in properties according to their size. This is especially true with regard to the surface tension, the viscosity, and the freezing point. These properties influence the performance of the soil mass so largely that the subject of soil physics almost completely resolves itself into a study of the physical properties of soil moisture.

Types of Soil Moisture.—Drops of water $\frac{1}{40}$ in. to about $\frac{1}{1,000,000}$ in. in size, suspended in air, appear as fog if one walks through them and as a cloud if viewed from a distance. Under electrical stress they coalesce to form raindrops ranging in size from about $\frac{2}{1,300}$ to $\frac{1}{4}$ in. This eventually becomes soil moisture, which may be of two kinds—adsorbed or free—as concerns its adhesion to soil particles and of four kinds—gravitational, capillary, cohesive, or solidified—depending upon the performance of the water or the properties that it imparts to the soil mass.

In colloidal suspensions, each soil particle may be considered as encased in a film of adsorbed water and surrounded by free water as illustrated in Fig. 30.

Free water has the freezing point, the boiling point, the surface tension, and the viscosity of ordinary water. Adsorbed-water films, in contrast, have higher boiling points, lower freezing points, greater surface tension, and are more viscous than free water. According to Terzaghi (84), they become semisolid substances at thicknesses less than $\frac{2}{1,000,000}$ in.

The outermost layer of an adsorbed film is most nearly like free water, and the innermost layer more nearly like solidified water or ice. Within the thickness of the film all the properties from those of free water to those of ice are then represented.

Thorough understanding of the effect of moisture on the performance of soil-water mixtures requires that the distinction between film moisture and pore or free water be kept in mind at

all times. In completely saturated soil, every particle is covered with a coating of film moisture attracted by the particle more strongly than by the force of gravity. Until enough water is added to the soil to satisfy the condition of equilibrium between attraction of gravity and of the soil particle for water, all the contained moisture is arranged around the particles as films, with the interstices between the films filled with air as shown later in Chap. XII.

The pore water, whether stationary or mobile, as referred to in this book is that which fills or flows through the interstices

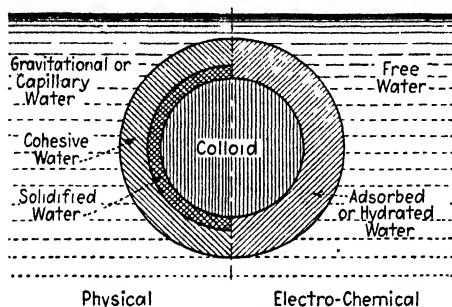


FIG. 30.—Types of soil moisture.

remaining between the outer surfaces of the adsorbed moisture films and not between the actual surfaces of the soil particles.

Therefore, all test results which depend upon the porosity of the soil are influenced by all variables which affect a change in the thicknesses of adsorbed films. Change of temperature alone, when all other conditions are kept constant may cause considerable variation in film thicknesses as shown later in Chap. XIII. Consequently, the test results commonly termed "constants" are truly constants only when the temperatures at which the tests are performed do not vary too widely.

Free or Gravitational Moisture.—Gravitational moisture is free water which enters soils owing to gravity during rains and thaws. It fills all the pore space of soils, loosely consolidated sediments and solid rock up to a certain definite level called the water table. The zone below the water table is known as the zone of saturation, and the water contained in it the ground water. The water table is not a level surface but rather a subdued replica of the land surface beneath which it lies. It is arched up under the hills,

dips slightly under the valleys, and intersects the surface in some low-lying tracts to form lakes, swamps, or springs. Ground water becomes very scarce below depths of several thousand feet, although exceptions occur where deep fissures or similar aquifers are encountered.

The region between the water table and the surface of the ground is called the zone of aeration. In regions where the main water table lies at a considerable depth below the surface an impervious stratum in the zone of aeration may hold a local zone

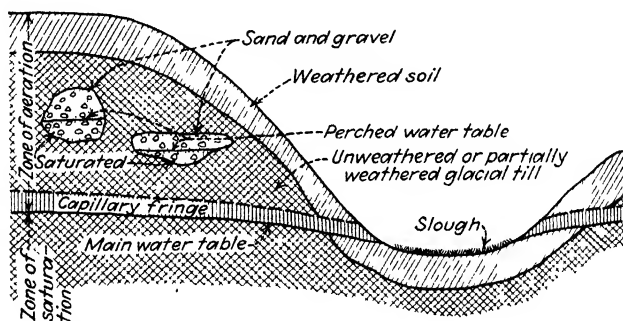


FIG. 31.—Soil profile illustrating main and perched water tables, zones of saturation and aeration, and capillary fringe.

of saturation suspended far above the main one. This zone has its own water table called a perched water table (Fig. 31).

Capillary Moisture.—The term “capillary moisture” as used in this text is defined as the free or pore moisture which adhesive films lift above the water table and which is held by its surface tension of free water at a distance, termed the “capillary rise,” above the water table. The layer of soil moistened by capillary moisture in the zone of aeration above the water table is termed the “capillary fringe” (22).

If any material, such as a piece of rock, a glass tube, or a steel needle, that has a greater affinity for water than for air is partly immersed in water, the water will, as a rule, be pulled slightly upward where it comes into contact with the inserted object, forming a meniscus (see Fig. 32). This may be explained as follows: Owing to the attraction of water for glass, a film of, possibly molecular thickness proceeds upward along a glass tube, wetting it for a distance above the liquid depending upon the magnitude of the attraction between the particular glass and the

water. This film was originally part of and is connected with the film or skin covering the surface of the water. Consequently, as it moves upward along the tube it tends to pull the adjacent water covering with it, thus causing the meniscus to be formed.

If, however, the glass tube is covered before immersion with a film of oil or other material which has a greater affinity for air than for water, there will be a film of adsorbed air which will prevent the water from wetting the oiled surfaces. Consequently, the adjacent water surface will be depressed in the form of an inverted meniscus, instead of elevated. It is because of the adsorbed film of air on the surface of the oil coating that needles so treated may

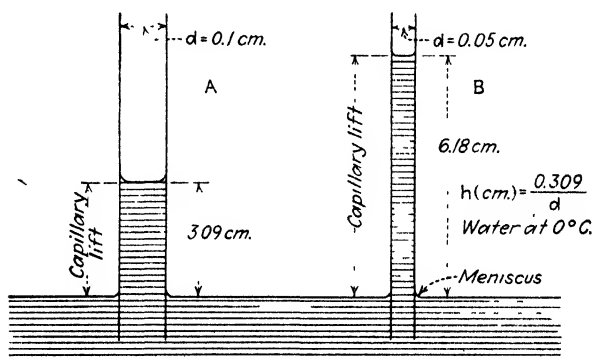


FIG. 32.—Diagram illustrating capillary lift in tubes of different diameters.

be made to float on the surface of water despite their greater density.

If a clean glass tube of very small diameter is held upright with one end immersed in water, the water will be drawn upward in the tube, filling it to some elevation above the surface of the water outside with the formation of a meniscus inside the tube also.

The height to which adsorbed moisture films rise above the meniscus on the clean glass surface and the depth to which the air film extends below the inverted meniscus in the case of the oiled tube depend upon the affinity between glass and water in the first case and between the oil film and air in the second. The maximum height to which the water will rise in the tube or rise or depress to form menisci adjacent to the tube is controlled by the surface tension or the strength of the film or skin with which the water is covered.

Factors That Determine the Height of Capillary Rise.—The effect of surface tension in limiting the height to which pore water may rise may be illustrated mathematically. The upward pull of a liquid in a cylindrical capillary tube is $3.1416dST \cos \alpha$; the downward pull is $0.7854d^2hD_1g$, where d = diameter of tube, centimeters.

ST = surface tension, dynes per centimeter.

α = angle of contact between the liquid and the walls of the tube.

h = height of capillary rise, centimeters.

D_1 = density of the liquid.

g = acceleration due to gravity, dynes.

These two forces are equal to each other. Therefore

$$h = \frac{4ST \cos \alpha}{dD_1 g} \quad (13)$$

The surface tension of water in a clean glass tube varies from $ST = 75.6$ dynes at 0°C. to $ST = 72.1$ at 25°C. Consequently, water at 0°C. would rise to a height of

$$h = \frac{4 \times 75.6 \times 1}{0.1 \times 1 \times 980} = 3.09 \text{ cm.}$$

in a tube of 0.1 cm. diameter, since $\cos \alpha$ and D_1 are both approximately equal to 1 at equilibrium. The height of rise at 0°C. for any diameter (see Fig. 32) would then be

$$h = \frac{0.309}{d} \text{ cm.} \quad (14)$$

Capillarity Influenced by Adsorption.—There has been considerable controversy as to whether or not the character of the wall has an effect on the height to which water will rise in capillary tubes. This is partly because glass tubes have been used in most of the experiments and the attraction between water and glass is very great. Because of this it is possible that in most cases surface tension is the controlling factor in the height of rise in clean glass tubes. With materials of less affinity for water than glass, where the maximum height of rise is not reached, adsorptive attraction may be the controlling factor.

Meinzer (22) calls attention to the work of Bigelow and Hunter who were apparently the first investigators to make direct measurements of the capillary range in tubes other than glass.

Glass tubes of the type *B* (Fig. 33) were used, only the covering plates with the apertures *a* being of other materials. These investigators concluded that the capillary ascension of water is different in tubes of different substances and that it is probably "a measure of the adhesion between the liquid and the substance in the wall."

However, there is not a complete agreement among investigators that water will rise to the same height in all capillary tubes even when the walls are made of glass. There is the indication that if the tubes are completely immersed and then lifted ver-

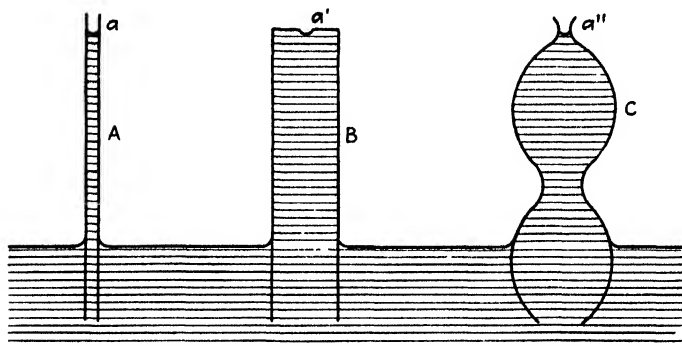


FIG. 33.—Diagram illustrating that capillary rise is dependent upon the size of orifice and not on size or shape of tube.

tically out of the water, the water columns gradually come to rest at about the same elevations in the tubes; but that if only the lower ends of thoroughly dried glass tubes are immersed, the water will rise to different heights and will not come to rest at the same elevation. Also, it is known that water will not rise so high in a dirty tube as in a clean tube.

An explanation which has been offered as to the reason for the capillary rise being equal when the glass tubes are first immersed is that a film of water is adsorbed on the surface of the wall so that the capillary moisture, instead of rising in a tube of glass, really rises in a tube of water. The attraction between the water columns and the moisture-film linings of the tubes is the same irrespective of the type of glass. Consequently, the height to which the capillary columns rise is the same. In the second case, when the glass tubes are first thoroughly dried and just the

ends are immersed, the effect of the relative attraction between the different glasses and the water is reflected in the height of rise. That there is a difference in the attraction of different types of glass is indicated by the observations of Pettijohn (68) referred to previously.

Capillary moisture is drawn up into the pores of permeable rock and soil just as in capillary tubes. The greater the attraction of the water for the solids the higher above the water table will moisture wet the surfaces of soil particles, and the thicker will be the capillary fringe. The high tensile strength of water causes columns of moisture to be drawn up into the soil as the surface-coating films, acting like thin elastic-rubber diaphragms, proceed along the walls of the soil's pores.

For equal pore size, the greater the adhesive attraction the higher will be the capillary rise, the maximum height being limited by the surface tension of water. For equal adhesive attraction, the height to which capillary moisture will rise increases as the diameters of the pores decrease. The force that lifts the moisture columns depends upon the cross-sectional areas of the pores. As the pores become smaller, the ratio of circumference to area increases. The lifting power thus increasing at a faster rate than the weight to be lifted accounts for the increasing height of capillary rise with decreasing pore size.

In soil work the deciding factor as to whether capillarity will prove detrimental or not is the speed as well as the height of capillary rise. While the latter increases, the former decreases with reduction in size of pore even when the surface tension remains more or less constant. However, as the size of the water particles gradually diminishes, a size is eventually reached at which the characteristics become those of film instead of free water.

Film or Adhesive Moisture.—When the capillary fringe does not extend to the surface of the ground, the moisture films which sustain the columns of moisture in the pores of the "fringe" penetrate the soil above and coat the particles in the same manner as the moisture films above the menisci wet glass, without, however, filling the pores. This has been termed "secondary" capillary moisture to distinguish it from that in the fringe, termed "primary" capillary moisture. The term "capillary moisture" used herein refers to the moisture in the fringe only.

The relative extent to which the free water and film water influence the performance of soils depends upon both the moisture content of the soil and the surface area of the soil particles. The lower the moisture content at equal grain size and the finer the soil at equal moisture content the greater will be the relative influence of the adsorbed water.

Consistency Limits.—The effect of moisture on soil performance may be explained as follows:

A small amount of soil may be mixed with water to form a soil suspension out of which, when set aside, the soil particles will gradually settle to the bottom of the container. The porosity, or moisture content, of the sediment at first is so high that it has all the properties of a fluid; *i.e.*, it will deform owing to its own weight to acquire the shape of any vessel in which it is placed and will have buoyant properties such that a hydrometer will indicate its density. This is the truly liquid state of the soil, the minimum moisture content of which is the flocculation limit.

The ratio of free to adsorbed water is now so large that the free water controls the stability of the soil. The pores are completely filled, and the soil mass is encased in the rubber-like film which exists on free water surfaces. There are no menisci; consequently no pressure is exerted by the enveloping surface film on the soil mass.

Reduction of moisture below the flocculation limit causes menisci to form, placing the enclosing film in tension, which, in turn, produces a uniformly distributed force acting perpendicularly to the outer surface of the sample at every point and tending to force the solid particles closer together.

So long as the mixture acts like slips and slurries which deform under their own weights but do not have the buoyancy of fluids, the soil is in the viscous state. The minimum moisture content of the soil in this state is the true lower liquid limit, although it is greater than the test constant, the liquid limit as determined by the Atterberg method described later.

With evaporation of moisture below the true lower liquid limit the soil mass increases in density and, like a plastic paste, no longer flows without the application of external force. A moisture content is eventually reached at which cohesive soil changes abruptly from a plastic to a rigid material, with correspondingly abrupt and considerable increase in bearing value.

A similar abrupt and considerable increase in bearing value occurs during the evaporation of moisture from slips and slurries of friable soils. The ratio of lubricating to adhesive moisture becomes small enough for the adhesive water abruptly to become the dominating influence on the performance of the soil.

The "critical" moisture content is the maximum moisture content of soils in the semisolid state. It was found by Levenson (85) to equal approximately the plastic limit of plastic soils and 75 per cent of the liquid limit of friable, fine-grained materials.

The contracting force increases as the sample becomes smaller, but increasing friction between particles causes the soil mass to

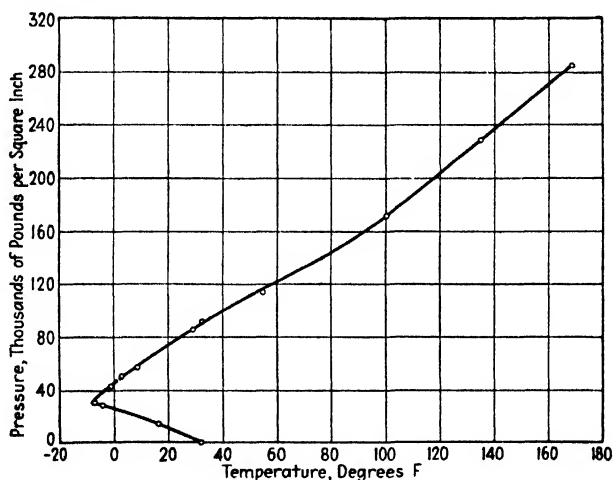


FIG. 34.—Effect of pressure upon the freezing point of water.

offer greater resistance to further reduction in volume. When the resistance of the soil to further consolidation just equals the contractive effort of surface tension a moisture content termed the "shrinkage limit" is reached, at which shrinkage ceases.

Below the shrinkage limit the soil is in the solid state. Further evaporation causes the mobile moisture to recede in the pores, leaving only the very thin, more or less solidified films to coat the soil particles. The soil then changes in color from dark to light but does not appreciably decrease in volume.

When finally all of the pore moisture is evaporated, the cohesive films are reduced to a solidified state with molecular thicknesses, under pressures estimated to exceed 20,000 atmospheres. Such

films are really ice, since at the enormous pressures under which they exist the melting point of ice may be, according to the Smithsonian tables (86), in excess of 76°C ., or 168°F . (see Fig. 34). It is because of these solidified-water films that thoroughly dried plastic soils form cakes and clods instead of dust. The strength of the clod increases generally with the paste-forming properties of the soil, the purely friable materials drying, as a rule, to powder.

Heat Produces Permanent Films.—If soils are dried at high instead of low temperatures, the formation of solidified-water films becomes more and more a permanent chemical action as the temperature increases until, when the temperatures used in pottery manufacture are reached, complete vitrification occurs. A. M. Wintermyer, in 1925 (87), observed a reduction in the plasticity of a clay soil when heated above 200°C ., indicating that adsorbed films had begun to assume a permanent aspect. Beginning with a slight decrease in the plasticity of the soil when heated at 300°C ., a still greater reduction occurred when the temperature was raised to 400°C ., and the plastic properties were entirely destroyed when a drying temperature of 500°C . was reached. Permanent change in the character of the soil was indicated also by changes from the normal color of the red-clay soil to a darker color at 300, a dark brick red at 400, and a shade yet darker at 600°C .

Shrinkage Due to Capillarity.—The loss in the volume of soil during shrinkage above the shrinkage limit is due entirely to and is proportional to the moisture loss. Thus for every cubic centimeter loss in volume there will be 1 gram loss in the weight of the soil. Because of this there is a direct relationship between weight and volume losses as shown in Fig. 35.

As long as the evaporating moisture has the properties of free water, the contractive force exerted by the shrinking water skin upon the surface of the drying soil may be estimated from the data used previously for computing the maximum height of capillary rise.

Assume the pores in the soil mass to be of square cross section and to have each a width equal to m (centimeter). The perimeter of each opening on the surface then equals $4m$. The force exerted by the surface tension of water at 0°C . is 0.07714 gram per centimeter, and therefore the force exerted upon each opening is equal to $4m \times 0.07714 = 0.309m$ gram.

If it is assumed that the soil surface is completely covered by such openings, the number of openings per square centimeter equals the reciprocal of the area m^2 of each opening. Consequently, if SF be designated as the force causing shrinkage in the soil,

$$SF = \frac{0.309m}{m^2} = \frac{0.309}{m} \text{ grams per square centimeter} \quad (15)$$

It can be readily seen that as long as the surface tension of water remains nearly constant, the capillary contractive force varies inversely as the size of the soil pores. Thus, for pores 0.1 mm. wide the force equals 30.9 grams per square centimeter,

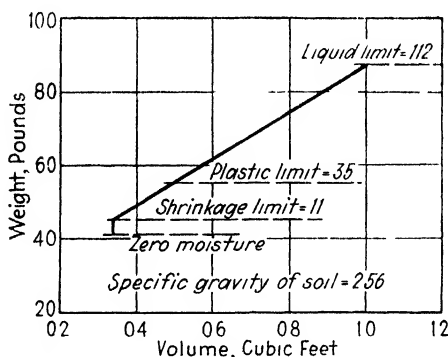


FIG. 35.—Relation between loss of weight and loss of volume in a soil-water mixture on drying.

and for pores 0.001 mm. wide it equals 3,090 grams per square centimeter. However, when the water films become so thin that the surface tension exceeds that normally possessed by water, the corresponding force productive of shrinkage increases enormously. Under these conditions the tension of the moisture may exceed several times the tensile strength of steel.

Swell of Soil Due to Film Moisture.—Moisture entering soils due to adsorption tends to separate the particles and causes the soil mass to swell. This may cause unconfined samples to lose stability and may produce enormous pressures in soil confined so that swell cannot occur. Capillary moisture following the adsorbed films into the soil may contribute to the loss of stability of some soils and be the principal agency which causes others to lose stability.

Samples of compressed wet soil resist disintegration to an enormously greater extent if they are immersed in water in the wet state instead of first being thoroughly dried. This may be explained as follows: The wet sample, if compressed sufficiently, has its particles completely covered with adhesive films, with the free water, if any is present, entrapped in the interstices, so that the whole is cemented into a dense mass impervious to the entrance of additional water.

In the dried samples, in contrast, the particles are coated with much thinner films and all the moisture is absent from the interstices between the films. Consequently, they are pervious enough for the water to enter and produce the swell productive of slaking.

The relative resistance of wet and dry soil samples to the entrance of capillary moisture is demonstrated by experiments reported in *Public Roads* 1931 (27).

Two disks cut from each of a number of compressed-soil samples were immersed in water—one in the wet state and the other after being dried to constant weight. Sixty-seven of the disks immersed in the wet state remained intact after being immersed for an average period of 9 months. The corresponding disks, immersed in the dry state, disintegrated after being immersed for periods ranging from 10 min. to 1 hr.

Effect of Swell on the Volume of Soil.—The relation of moisture content to volume change during swell differs from that which obtains during shrinkage of the soil.

If a powdered clay is completely dried to constant weight in an oven and then placed in the air, it will absorb moisture (termed the "hygroscopic moisture") up to some percentage generally not exceeding 10, depending upon the character of the soil material and the humidity of the air.

Absorption of the hygroscopic moisture may cause the soil, if not confined, to undergo a considerable increase in volume which may be subsequently decreased by gradually adding more moisture to the soil.

In sands, the increase in volume at hygroscopic moisture content of 6 to 8 per cent is termed "bulking" and may amount to as much as 25 per cent.

The effect of increase of moisture content on the density of soil samples compressed in small brass molds at equal pressures is shown later in Chap. XII.

Tests Disclose Effect of Moisture in Soils (27).—The liquid limit is defined as the moisture content at which 10 light shocks produced by striking the dish against the hand will just close the



Divided Soil Cake Before Test



Soil Cake After Test

FIG. 36.—Diagram illustrating liquid-limit test.

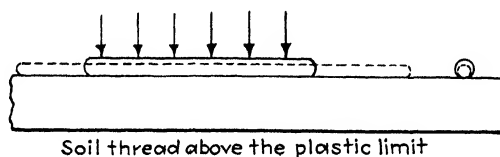
groove in a soil sample such as is shown in Fig. 36.

The plastic limit is defined as the lowest moisture content at which the soil can be rolled into a thread $\frac{1}{8}$ in. in diameter without the thread's breaking into pieces (see Fig. 37).

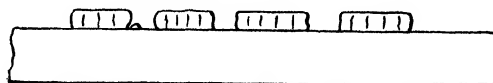
The numerical difference between the liquid and the plastic limits, defined as the plasticity index, shows the range in moisture contents

through which the soil remains plastic.

The shrinkage limit is defined as the moisture content at which further evaporation of moisture does not produce further decrease in the volume of the soil.



Soil thread above the plastic limit



Crumbling of soil thread below the plastic limit

FIG. 37.—Diagram illustrating plastic-limit test.

In the shrinkage test, soil powder and water are mixed into a paste slightly above the liquid limit, placed in a porcelain milk dish, as shown in Fig. 38, and then dried to constant weight. While drying, the sample loses volume proportionately to the moisture loss above the shrinkage limit as shown in Fig. 39.

Here the volume change of the muck soil equals 335 per cent when the moisture content is reduced from 269 to 33 per cent. Consequently, the ratio of volume change to moisture loss,

termed the "shrinkage ratio" (Fig. 39), equals

$$\frac{335}{269 - 33} = \frac{335}{236} = 1.42$$

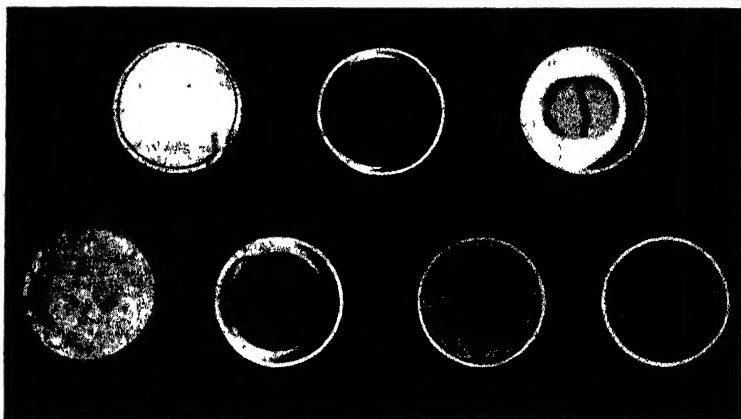


FIG. 38.—Shrinkage of representative soil constituents. Top row, left to right: diatoms, peat, colloids. Bottom row: mica, clay, silt, sand.

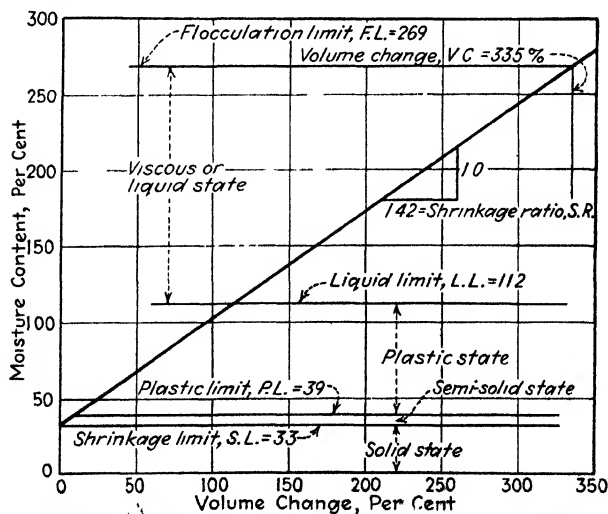


FIG. 39.—Relation between loss of moisture and loss of volume of muck soil on drying.

The shrinkage limit, the shrinkage ratio, and the specific gravity are interrelated as follows:

When SL = shrinkage limit, moisture content, per cent.

w = moisture content of wet soil, per cent.

V = volume of wet-soil cake, cubic centimeters.

V_0 = volume of dry-soil cake, cubic centimeters.

W = weight of wet-soil cake, grams.

W_0 = weight of dry-soil cake, grams.

SR = shrinkage ratio.

G = specific gravity of soil particles.

$$\text{Shrinkage limit } SL = w - \frac{V - V_0}{W_0} \times 100 \quad (16)$$

$$\text{Shrinkage ratio } SR = \frac{W_0}{V_0} \quad (17)$$

$$\text{Specific gravity } G = \frac{1}{\frac{1}{SR} - \frac{SL}{100}} \quad (18)$$

The volume change VC which occurs when a soil sample is dried from any moisture content w is given by the formula

$$VC = (w - SL)SR \quad (19)$$

$$VC = \frac{w - SL}{\frac{1}{G} + \frac{SL}{100}} \quad (20)$$

This for the sample of muck soil becomes

$$VC = (w - 33)1.42$$

SR is also the bulk specific gravity G_0 of the soil. Thus the weight of a cubic foot of the muck soil in the dried state equals $62.4 \times 1.42 = 89$ lb.

Moisture Equivalents.—The moisture-equivalent tests indicate compressive and expansive properties.

The centrifuge moisture equivalent is defined as the moisture content of the soil when subjected to a centrifugal force of one thousand times the force of gravity for 1 hr. Essentially, the test (see Fig. 40) consists of first soaking a small sample of air-dried soil in a Gooch crucible, then draining it in a humidifier for at least 12 hr., and finally centrifuging it for 1 hr.

The centrifugal action has a twofold effect, *viz.*, (a) to consolidate the soil particles, forcing water toward both top and bot-

tom of the sample and (b) to expel water through the perforations in the bottom of the crucible. This combination of forces

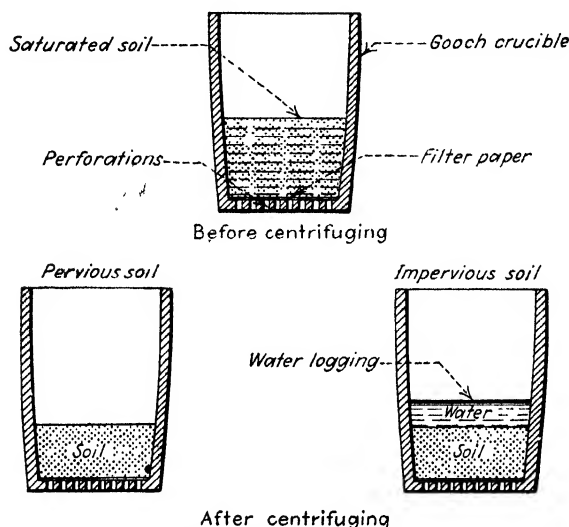


FIG. 40.—Diagram illustrating centrifuge moisture-equivalent test.

produces an average pressure on the sample of about 2 kg. per square centimeter. All free water is expelled from pervious samples. At least part of the water forced to the top of impervious samples is retained there to produce what is termed "waterlogging."

Field moisture equivalent is defined as the minimum moisture content at which a drop of water placed on a smoothed surface of the soil will not be immediately absorbed but will instead spread out over the surface and give it a shiny appearance (see Fig. 41).

Essentially the test consists of thoroughly mixing small increments of water with air-dried soil and smoothing off with a spatula until additional water, as referred to above, will not be absorbed.

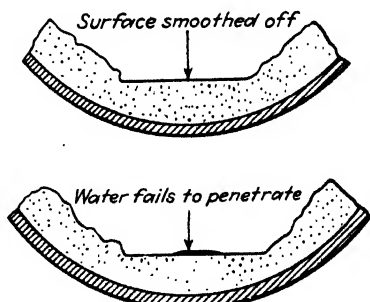


FIG. 41.—Diagram illustrating field moisture-equivalent test.

At this moisture content the expansive properties of the soil sample have been satisfied.

Specific gravities are determined by a method that includes boiling a sample of soil with water in a pycnometer and making computations by the formula

$$\text{Specific gravity} = \frac{c - a}{(b - a) - (d - c)} \quad (21)$$

where a = weight of dry bottle.

b = weight of bottle full of distilled water.

c = weight of bottle and weight of dry soil.

d = weight of bottle filled with soil and water.

The volumetric change is defined as the volume change in percentage of the dry volume which occurs when the moisture content of the sample is reduced from the field moisture equivalent to 0. It is computed from formula (19) by substitution of the field moisture equivalent FME for w . The volumetric change when designated by VC_f becomes

$$VC_f = (FME - SL) \times SR \quad (22)$$

The lineal shrinkage LS (6), expressed as a percentage of the length of a wet soil sample, is defined by the formula

$$LS = 100 \left[1 - \sqrt[3]{\frac{100}{VC_f - 100}} \right] \quad (23)$$

The relative volumes of the muck soil at the different limits of consistency are shown in Table 10.

Thus 100 cu. ft. of the muck soil at the flocculation limit becomes 49 cu. ft. at the liquid limit ($21\frac{1}{2}/435 \times 100$), 25 cu. ft. at the plastic limit, and 23 cu. ft. at the shrinkage limit.

TABLE 10.—RELATIVE VOLUME OF MUCK SOIL AT DIFFERENT LIMITS OF CONSISTENCY

Consistency	Moisture content, per cent	Volume change when dried to constant weight, formula (19) per cent	Relative volume in percentage of volume of dried soil
Flocculation limit..	269	335	435
* Liquid limit....	112	112	212
Plastic limit.....	39	9	109
Shrinkage limit....	33	0	100

Plastic Force Indicated by Flow Curves.—The plasticity index indicates only the range of moisture contents through which a soil has the properties of a plastic material. To complete the picture of the plastic characteristics of a soil, there should be information also on the degree of plasticity, which is indicated by the force required to deform the plastic soil. This information is furnished by flow curves which are obtained as part of the procedure used in determining the liquid limit by the mechanical instead of the hand method.

The apparatus (see Fig. 42) for making this determination was developed by Arthur Casagrande (88). It consists of a brass dish and cam mounted on a hard rubber block and arranged so that the divided soil cake can be subjected to shocks produced by fall of the dish through a constant distance and applied at a specified rate.

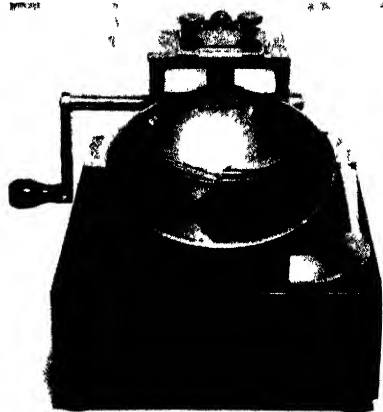


FIG 42 — Mechanical liquid-limit device

In the hand method of test previously referred to, water or soil powder as necessary is added to the test sample until the desired consistency is obtained. In the mechanical method, however, the number of shocks required to close the groove in the soil at different trial consistencies is determined. The relation between shocks and moisture content is plotted to form what is known as the flow curve, which is a straight line on a semilog plot and is expressed by

$$F = \frac{H - w}{\log M} \quad (24)$$

in which w = moisture content at M shocks.

The flow index F is the range in moisture content represented by the number of shocks in one cycle of the logarithmic scale. The intercept H is the moisture content at one shock on the curve. For the muck soil (Fig. 43), $H = 152$ per cent, and $F = 28.4$. Consequently, for this sample,

$$w = 152 - 28.4 \log M$$

The liquid limit as determined by the mechanical method is the moisture content at 25 shocks on the flow curve, as compared with 10 shocks by the hand method. The mechanical device could be adjusted so that 10 shocks would indicate the liquid limit on the flow curves also. However, this would only increase the wear and tear on the apparatus without benefiting the test.

On the assumption that the shocks required to close the groove are an index of shear resistance, the flow curves indicate the

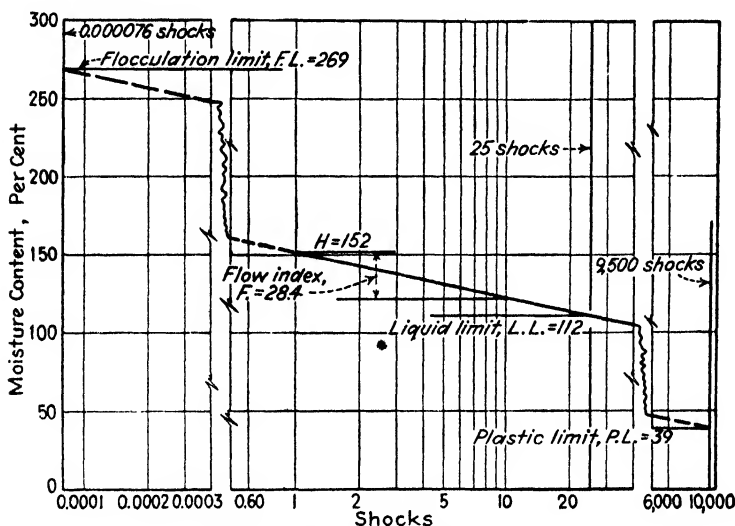


FIG. 43.—Flow curve of muck soil.

influence of change of moisture content upon the force required to deform plastic soils. Also, if the shocks at the plastic limit on the flow curve can be taken as a measure of the toughness of soil, the number of logarithmic cycles included within the range of moisture contents comprising the plasticity index PI on the flow curve can be used as a toughness index TI . It is expressed as

$$TI = \frac{PI}{F} \quad (25)$$

Problems

1. With respect to its adhesion to soil particles, name the kinds of soil moisture and state the differences between them.
2. What is meant by gravitational water; water table; perched water table?

3. What is capillary moisture and what is responsible for the rise of capillary moisture in soils?

4. What factors influence the height to which capillary moisture will rise in soils?

5. Discuss what takes place when a soil mass changes from a slurry to the solid state.

6. When all of the pore moisture is evaporated, how does a friable soil differ from a plastic soil? Explain.

7. What relation exists between moisture loss and volume change in a soil above the shrinkage limit?

8. What is the magnitude of the force producing shrinkage in a soil whose pore diameter is 0.005 mm.?

9. Define (a) liquid limit, (b) plastic limit, (c) plasticity index, and (d) shrinkage limit.

10. A soil has a shrinkage limit of 18 and a shrinkage ratio of 1.80. Compute the specific gravity of the soil particles and the volume change when the soil is dried from a moisture content of 48 per cent.

11. (a) Define centrifuge moisture equivalent and field moisture equivalent.

(b) Describe briefly how the tests are performed.

12. The soil referred to in Prob. 10 has a field moisture equivalent of 30. Compute the volumetric change and the lineal shrinkage.

13. Describe briefly the steps involved in determining the flow index of a soil.

14. What is the toughness index of the soil whose flow curve is shown in Fig. 43?

CHAPTER VI

MOVEMENTS OF SOIL MOISTURE

Flow of moisture through soils results from a complex combination of influences. There are involved the speed of free water movement, the height of capillary rise, the elevation of the water table, and (as discussed later in Chap. XIII) the temperature of the soil.

Permeability of Soils.—Permeability is defined as that property of soils which permits the passage of moisture, solutions, or similar fluids. The passage of moisture through the interstices or pores of the soil is termed “percolation.” Soils porous enough for percolation to occur are termed “pervious” or “permeable.” Those which do not permit the passage of fluids are termed “impervious” or “impermeable.”

In dry soils the flow is due to capillarity's attempting to satisfy the adsorptive attraction of moisture for the surface of the soil particles, supplemented by the percolation of moisture through the soil pores due to gravity.

As the adsorptive attraction of the soil becomes more and more satisfied by the penetrating moisture, the permeability depends more and more on the action of gravity until with complete saturation of the soil all the flow is percolation due to gravity.

During rains and thaws, part of the water flows away on the surface of the ground, and part enters the soil and percolates vertically downward until it meets the water table. Below that level the ground water proper moves in the same general direction as the surface runoff but more slowly, because the friction caused by its passage through the interstices of the soil retards its motion.

Wherever the surface of the ground, because of either natural depressions or artificially constructed ditches and trenches, intercepts the water table, ground water emerges in some fashion, such as springs, to form swamps or to flow into drains. Immediately adjacent to where the ground water emerges, the water table is depressed. Were it not for the restraining influence of friction, the subsurface flow would be as rapid as that at the surface, and

the zone of saturation would be no thicker than that of surface runoff of equal volume. As a result, the depressed water tables adjacent to drains would be eliminated. Therefore, the character of the water table adjacent to drains becomes indicative of the frictional resistance of the soil. Thus the greater the depression of the water table the higher the frictional resistance of the soil

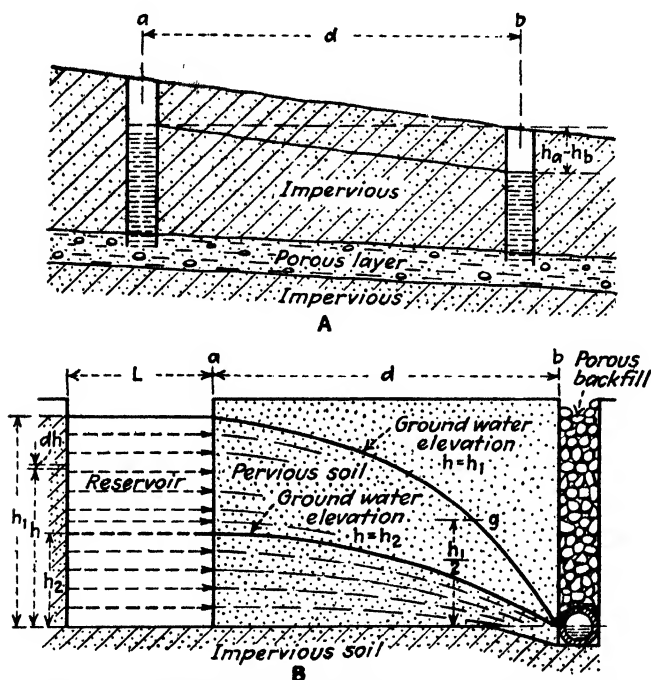


FIG. 44.—Diagram illustrating flow of moisture through soil.

is indicated, and, conversely, the less the depression the lower the frictional resistance.

Hydraulic Gradient.—The term "hydraulic gradient" is used in drainage computations to indicate the magnitude of the frictional resistance of the soil to percolation. It is the quotient obtained by dividing the drop h in head which occurs in the percolating water between two points in a saturated-soil column by the distance d between the points.

If the permeable soil layer is located between two impervious layers, as shown in Fig. 44A, the difference in head is disclosed by

observing the height to which water rises in two wells a and b which extend to the "aquifer," or water-bearing stratum. In this case the hydraulic gradient equals the difference between the pressure head h_a at a (in feet); and h_b at b divided by the distance d (in feet), the distance between a and b . In this case $\frac{h}{d} = \frac{h_a - h_b}{d}$. Here the term has the significance commonly accepted in hydraulics.

When, however, there is no impervious layer on top of the porous layer the hydraulic gradient equals the difference in the elevation of the ground water at two points in a soil layer divided by the horizontal distance between them (see Fig. 44*B*). The hydraulic gradient may be considered to equal also the drop in head between two faces of a soil cake divided by the distance, horizontal or vertical, between the faces. When the drop in head in the wells in the first case, between the two points in the soil layer in the second, and between the faces of the soil cake in the third just equals the respective distances, the hydraulic gradient in each case is said to equal 1, or unity.

Two terms, the coefficient of percolation k_p and the coefficient of permeability k , are employed to denote the speed or rate of percolation.

Coefficient of Percolation.—The coefficient of percolation k_p is defined as the velocity of flow through the pores of the soil under a hydraulic gradient of 1. The coefficient of permeability k , in contrast, is defined not as the actual velocity of flow through the soil pores but instead as the velocity with which flow would occur through a cross-sectional area of the soil equal to that occupied by both pores and solids. Hence, k_p is larger than k by an amount depending upon the size of the pore area available for moisture transfer.

If in a soil mass the pore ratio $e = 0.7$, then in a cross section of 1 sq. ft. of soil mass the pore area would equal

$$\frac{0.7}{1 + .7} \times 1 = 0.412 \text{ sq. ft.,}$$

which is the area through which the velocity k_p occurs. The corresponding coefficient k , however, is the velocity that would occur if the same amount of water traveled during the same period of time through a cross-sectional area of 1.0 instead of 0.412 sq. ft.

Thus

$$1 + e \times k_p \quad (26)$$

If in the present instance $k_p = 15$ ft. per day, k will become

$$\frac{0.7}{1 + 0.7} \times 15 = 6.18 \text{ ft. per day}$$

For a cross-sectional area of unity, both k_p and k denote discharge as well as velocity. Thus, k_p of 15 ft. per day or k of 6.18 ft. per day both denote for each square foot of cross section of soil mass a discharge of 6.18 cu. ft. per day.

Derivation of Coefficients of Flow.

According to Darcy, the velocity v of water percolating through soils equals the product of the hydraulic gradient into the unit velocity at a hydraulic gradient of 1. It is expressed by the formula

$$v = k_p \frac{h}{d} \quad (27)$$

Where h = difference in the elevation of the ground-water level at two locations in a soil layer.

d = horizontal distance between the locations, thus making

$\frac{h}{d}$ = the hydraulic gradient.

k_p = velocity of flow through the soil pores when $h = d$.

From this the coefficient of permeability k is derived in the following manner based on work by Glennon Gilboy (89.)

Let a = cross-sectional area of standpipe (Fig. 45).

A = cross-sectional area of soil column.

d = thickness of soil column.

E_t = elevation of overflow from the soil column, which is constant.

If the addition of Q cc. of water is required to maintain the elevation of water in the standpipe at constant elevation E_1 for a

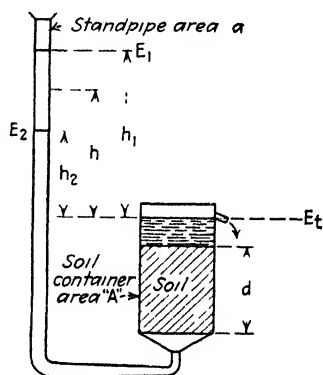


FIG. 45.—Permeameter for determining coefficients of flow.

period of time t sec.; and if the soil mass has a thickness of d and a pore ratio of e , then

$$\frac{h_1}{d} = \text{hydraulic gradient.}$$

k_p = velocity of flow through the soil pores.

k = coefficient of permeability = the velocity through the container above the soil mass.

$$\begin{aligned} Q &= k_p \frac{h_1}{d} t A \frac{e}{1+e} \\ &= k \frac{h_1}{d} t A. \\ k &= \frac{Qd}{tAh_1} \end{aligned} \quad (28)$$

If the following numerical examples are substituted,

$$Q = 2 \text{ cu. ft.}$$

$$t = 24 \text{ hr.}$$

$$h_1 = 8 \text{ ft.}$$

$$d = 3 \text{ ft.}$$

$$A = 1.2 \text{ sq. ft.}$$

$$e = 0.8.$$

$$k = \frac{2 \times 3}{24 \times 1.2 \times 8} = 0.02604 \text{ ft. per hour.}$$

$$k_p = \frac{1 + 0.8}{0.8} \times 0.02604 = 0.05859 \text{ ft. per hour.}$$

If no water is added, the elevation in the standpipe will drop as the water percolates through the soil column. In a period beginning at t_1 and ending at t_2 the drop will be from h_1 to h_2 . In this case let h equal the head at any time t . Then in the next succeeding interval of time dt the elevation of the water in the standpipe will drop a distance dh . The volume of water lost from the standpipe equals $-a dh$. This is the amount of water that passes upward through the soil container during the time dt , which equals $k \frac{h}{d} A dt$. Therefore,

$$-a dh = k \frac{h}{d} A dt \quad (29)$$

This equation may be integrated between the limits h_1 and h_2 for h , and t_2 and t_1 for t , giving the following general value of the coef-

ficient of permeability:

$$k = \frac{ad}{A} \frac{1}{t_2 - t_1} \log_e \frac{h_1}{h_2} \quad (30)$$

Capillary Flow of Water in Soils.—When the capillary fringe extends to the surface of the ground, and if the ground surface be waterproofed so that no evaporation can occur, the soil is said to be in capillary equilibrium, a condition under which there is no flow of capillary moisture. When, however, there is evaporation from the surface, capillary equilibrium is destroyed, and moisture moves upward from the water table to replace that which is lost from the surface. Under these conditions there is a continuous flow of capillary moisture upward toward the surface of the ground. It has been estimated that in order to sustain a field of growing corn in the Middle West, as much as 0.9 lb. of water per square foot of soil area has been raised through a capillary fringe 4 ft. in thickness in 24 hr. At this rate a layer of water approximately $5\frac{1}{2}$ ft. in depth would be raised during one year.

It is to be emphasized that while the rise of water through the dry soil is due to capillarity, the flow required to replace the moisture lost from the top of the layer is accomplished by the high tensile strength of water. This has been estimated as great enough to sustain a vertical column as long as 2,000 ft.

Flow of Gravitational Moisture.—Let it be assumed that the pervious soil bank (Fig. 45*B*) consists (a) of sand with a coefficient of permeability k equal to 175.6 ft. per day and (b) of silt with k equal to 6.34 ft. per day; that the specific gravity of both the sand and the silt is 2.65; that the sand has a capillary rise of 0.4 ft. and a pore ratio of 0.54, of which 0.015 is capillary moisture; and that the silt has a capillary rise of 12.0 ft. and a pore ratio of 1.06 of which 0.96 is capillary moisture. Also let dimensions be assumed as follows:

$$h_1 = 15 \text{ ft.}$$

$$h_2 = 3 \text{ ft.}$$

$$d = 50 \text{ ft.}$$

$$L = 20 \text{ ft.}$$

Let it be required to find (A) the discharge per day from the reservoir, the water level remaining constant at height h_1 ; and (B) the time required for the water level to drop from the height h_1 to h_2 .

Case A.—The hydraulic gradient in this case equals h_1/d . The velocity v in feet per day through the vertical face of the reservoir is given by the equation

$$v = k \frac{h_1}{d} \quad (31)$$

The discharge Q in cubic feet per day through a strip of the vertical face m ft. wide is denoted by the equation

$$Q = \frac{mkh_1^2}{d} \quad (32)$$

Substituting numerical values in Eqs. (31) and (32), we have for the sand embankment

$$v = 175.6 \times \frac{15}{50} = 52.68 \text{ ft. per day}$$

$$Q = \frac{175.6 \times 15 \times 15}{50} = 790.2 \text{ cu. ft. per day per foot width}$$

For the silt embankment we have

$$v = 6.34 \times \frac{15}{50} = 1.902 \text{ ft. per day}$$

$$Q = \frac{6.34 \times 15 \times 15}{50} = 28.53 \text{ cu. ft. per day per foot width}$$

The velocity v as used in the foregoing calculations refers to the rate of flow through the reservoir section toward the face of the bank and is constant. The rate of flow through the soil is greater than v and is variable, increasing with increase in the distance from the face of the reservoir owing to the reduction in the section through which flow occurs.

The rate of flow through the soil at the face of the reservoir is equal to $\frac{e+1}{e} \times v$. At a point such as g where the section through which flow occurs is, say, one-half that at a , the rate of flow would be twice that at a .

Since the velocity v varies as the hydraulic gradient, for equal loss in head the lengths of sand and silt banks for which v would be equal will vary directly as the coefficients of permeability. In the cases cited, the length of a sand bank into which the velocity of flow would be the same as for a silt bank 50 ft. long would equal

$$\frac{50 \times 175.6}{6.34} = 1,385 \text{ ft.}$$

Case B.—To determine the time t in days required for the water-surface elevation to drop from h_1 to h_2 , let it be considered that the water drops a distance dh in a time dt . Then the hydraulic gradient equals h/d .

The amount of water that enters the face of the soil bank during any interval of time dt is that shown by the expression

$$dQ = \frac{m k h^2}{d} dt$$

But in the time dt the volume of water lost from the reservoir equals the product of the drop dh and the area of the reservoir. Thus we have also the relationship

$$dQ = -Lm \, dh$$

in which m is the entire width, and L the length of the reservoir perpendicular to the embankment. Then

$$-Lm \, dh = \frac{m k h^2}{d} dt$$

from which

$$t = -\frac{Ld}{k} \int_{h_2}^{h_1} \frac{dh}{h^2}$$

By integration

$$\begin{aligned} t &= \frac{Ld}{k} \left[\frac{1}{h} \right]_{h_1}^{h_2} \\ t &= \frac{Ld(h_1 - h_2)}{k h_1 h_2} \end{aligned} \quad (33)$$

By substitution of the numerical values the times required for the water level to drop from h_1 to h_2 are as follows:

For the sand

$$t = \frac{20 \times 50(15 - 3)}{175.6 \times 15 \times 3} = 1.5 \text{ days}$$

For the silt

$$t = \frac{20 \times 50(15 - 3)}{6.34 \times 15 \times 3} = 42.1 \text{ days}$$

These simple computations indicate that if drains are to be provided such that the time of removal of gravitational flow will be equal in different types of soil, either the horizontal spacing

of the drains must vary in inverse proportion to the coefficients of permeability, the depth of the drains remaining constant, or the depths of the drains must vary inversely as the coefficients of permeability, the horizontal spacing remaining constant.

However, the capillary moisture must be taken into consideration. Below the water table, in the case of the sand, the moisture

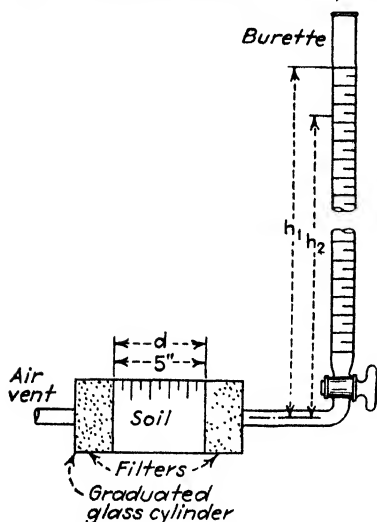


FIG. 46.—Combined permeameter and capillometer.

content is 20.4 per cent by weight of the dry sand; and above the water table the capillary fringe with a moisture content of 0.6 per cent extends for 0.4 ft. In the silt the moisture content below the water table is 40 per cent, and the capillary fringe extends upward from the water table for a distance of 12 ft. with a moisture content of 36 per cent. To be effective in soils of high capillarity, therefore, drainage must serve to lower the capillary fringe as well as to remove the free water.

Relative Effect of Capillarity and Gravity.—

For soil with the pores completely filled with moisture a number of relationships between rate of flow and porosity, grain size, etc., have been derived. As far back as 1892 Whitney (2) published formulas for relative flow based on grain size and also a method for determining the flow through undisturbed samples of soil. The more recent developments have been reported by Terzaghi (90) and Cox (91).

A typical equation suggested by Hatch is

$$k = 150 \times es^2 \times \left(\frac{E}{45} \right)^6 \times \frac{(T + 10)}{(60)} \quad (34)$$

in which k = the coefficient of permeability, meters per day.

es = effective size, millimeters.

E = percentage of voids.

T = temperature, degrees F.

In dams, subgrades, soil-road surfaces, and the like the soil as a rule is only partially saturated. Since the degree of saturation controls the extent to which the capillarity is satisfied, it can be said that the permeability that controls the rate at which soils soften depends upon the moisture content as well as the porosity. This is demonstrated by Table 11 which shows coefficients of permeability of various materials determined at both partial and complete saturation (see Fig. 46).

The velocity at partial saturation noted as water at start of test (Table 11) is due to both head and capillary attraction. At complete saturation, noted as water at end of test, the flow is due only to head. The difference is the velocity due to capillarity (62).

TABLE 11.—PERMEABILITY OF SOIL SAMPLES AT DIFFERENT MOISTURE CONTENTS

Sample number	Volume of solids, per cent	Volume of water present, per cent		Permeability k , feet per day		
		Start of test	End of test	Capillary	Gravity	Total
1	54	0	36	0.163	0.027	0.190
1	56	36	43	0.032	0.023	0.055
2	40	0	49	0.768	0.082	0.850
3	39	0	51	0.368	0.029	0.397

No. 1: silt loam soil; No. 2: ground quartz, 80 per cent; diatoms, 20 per cent; No. 3: ground quartz, 76 per cent; diatoms, 19 per cent; bentonite, 5 per cent.

The total velocity in the silt loam (sample 1) when dry is 0.190 ft. per day of which 0.163 ft., or 86 per cent, is due to capillarity and but 0.027, or 14 per cent, to gravity. When partially saturated, however, the total velocity is but 0.055 ft. per day, and 0.023 ft., or 42 per cent, is due to gravity. The total velocity in sample 2, representative of soils having high capillary properties, is 0.850 ft. per day, of which slightly less than 10 per cent is due to gravity. The effect of the presence of cohesive moisture is shown by sample 3. The small admixture of bentonite did not cause the porosity to vary appreciably from that of sample 2. The total velocity, however, is reduced to 0.397 ft. per day, of which slightly less than 8 per cent is due to gravity.

Table 12 furnishes additional information of this character. The difference between the drainable, non-frost-heaving beach sand and an unstable, frost-heaving silt is quite evident. Here it is indicated that in a 10-ft. layer of soil the thickness occupied by the capillary moisture in the silt is 4.7 ft., while that in the sand equals but one-tenth of 1 ft. This leaves in the sand an equivalent of 3.4 ft. of space for the flow of gravitational water in contrast to but 0.4 ft. in the silt. As would be expected, this difference is reflected in the gravitational flow which in the sand varied from 22.5 to 65.7 ft. in 24 hr. where for equal heads the flow in the silt varied from but 0.1 to 2.4 ft. in the same time. Furthermore, the absence of capillary flow in the sand is conspicuous.

TABLE 12.—DRAINAGE PROPERTIES OF A SAND AND A SILT SOIL COMPARED

Drainage properties		Beach sand	Silt soil
		Feet	Feet
Composition of 10 ft. of saturated soil	{ Soil particles	6.5	4 9
	{ Capillary moisture	0.1	4.7
	{ Gravitational moisture and air	3.4	0 4
Capillary rise per 24 hr.	{ 1.5 ft. above ground-water elevation	0.0	5 2
	{ 3.0 ft. above ground-water elevation	0.0	3.0
	{ 6.0 ft. above ground-water elevation	0.0	0 9
Maximum capillary rise		0.4	12.0
		Cubic feet	Cubic feet
Capillary flow per 24 hr. per square foot.	{ 1.5 ft. above ground-water elevation	2.7
	{ 3.0 ft. above ground-water elevation	2.0
	{ 6.0 ft. above ground-water elevation	1.0
Gravitational flow per 24 hr. per square foot.	{ 1.5 ft. head	22.5	0.1
	{ 3.0 ft. head	39.7	0.4
	{ 6.0 ft. head	65.7	2.4

Capillarity May Be Supplemented by Gravity.—Under certain conditions flow originated entirely by capillarity may be continued by gravity, as shown in Fig. 47. Capillarity draws the liquid up

through the cheesecloth wicks, over the edge of the container, and down to the ends of the wicks on the outside of the container. This merely moistens the wick on the left whose outside end is just about on a level with the surface of the liquid in the container. In addition to becoming moist, the wick on the right, the outside end of which is located appreciably below the surface of the liquid in the container, performs like a siphon and transfers the liquid from the upper to the lower container.



FIG. 47.—Illustration of combined capillary and gravitational flow.

The higher the humidity the less is the evaporation from the wicks, and as a result the greater is the amount of free or gravitational water transferred, being as much as six times as much on humid as on dry days. This difference in the rate of flow reflects the difference in the rate of

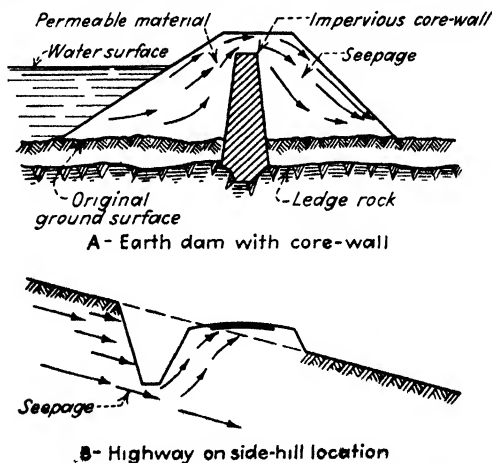


FIG. 48.—Illustrations of seepage and capillary flow in soils.

evaporation and serves to explain the discomfort suffered on hot days when the humidity rises. The rate at which the body is cooled depends not upon the rate at

which moisture emerges from the pores of the skin but upon the rate at which the moisture is evaporated. Therefore in the environment under which this experiment was performed, the rate of cooling on humid days was indicated to be as little as one-sixth as great as that on dry days.

This simple experiment explains how capillarity may produce water movement detrimental to structures in spite of preventive measures. Figure 48A shows how seepage may occur in the lower face of an earth dam, even though the impervious core wall is higher than the adjacent water surface. The water rising over the core wall, due to capillarity, may, after satisfying the capillary requirements of the soil in the lower face of the dam, flow out owing to gravity. If the seepage occurs at a lower elevation than the surface of the water on the upper face of the dam, a siphon-like action may be produced, and the flow will be considerable in amount. A very common case of water entering the subgrade of a road in spite of cutoff ditches is illustrated in Fig. 48B.

Problems

1. Define permeability; percolation.
2. What is meant by "hydraulic gradient" when used in drainage computations? Illustrate.
3. What is the difference between the coefficient of percolation and the coefficient of permeability?
4. What relation exists between the velocity of flow and the coefficients of percolation and permeability?
5. What is meant by capillary equilibrium?
6. In attempting to remove water from soils, what effect does the permeability of the soil have upon the spacing of drains; upon the depth of drains?
7. What relation exists between the capillarity and drainability of a soil?

CHAPTER VII

FROST PHENOMENA

Next to flow of water through soil, the phenomena responsible for the detrimental effects of frost seem to be least understood. Here, again, a number of influences are involved, among which are variations in air temperature and the speed at which they occur; the type of soil; the character and thickness of road surface; and the amount of water present.

Nature is engaged continuously in processes tending to stabilize materials of the earth's crust and create equilibrium by means of phenomena which diminish or completely eliminate differences in the state or the character of component or adjacent parts. Geological processes work to level off the earth's surface by lowering the mountains and filling up the valleys. Pedological influences serve by a process of seasoning to provide the exposed soils with a stability that prevents change of state under changing weather conditions.

Continuation of animal life requires that the sustaining elements be extracted from the stable inorganic compounds of the soil and changed into the unstable organic substances which serve as food. This is accomplished by means of the complex botanical processes. However, as soon as such compounds have served their purpose, they are quickly changed by putrefactive processes back again into the stable inorganic materials.

Heat Transfer.—Frost phenomena also are the result of efforts to create equilibrium—but of temperature in this case. There is a constant exchange of heat between closely associated substances which vary in temperature. The flow is from the warmer to the colder materials, just as the flow of liquids is from the higher to the lower levels.

The unit of heat is the "gram-calorie" in the c.g.s. system and the "British thermal unit" (B.t.u.) in the f.p.s. system. Sometimes the term "large calorie" is used to designate 1,000 gram-calories. The gram-calorie, which is usually designated by the single term "calorie," is defined as the quantity of heat

required to raise the temperature of 1 gram of water from 3.5 to 4.5° on the Centigrade scale. The mean British thermal unit is defined as the heat required to raise the temperature of 1 lb. of water from 60 to 61° on the Fahrenheit scale. One calorie = 4.18 joules = 4.18×10^7 ergs = 0.0039685 B.t.u.

The rate at which heat is conducted from the warmer to the colder substance is disclosed by the coefficient of thermal conductivity. It is defined as the quantity of heat transmitted per unit of time through a plate of unit area and unit thickness when the thermal gradient is unity. The thermal gradient is defined as the ratio of the drop in temperature between points in a wall to the distance between the points.

Thermal conductivity in the c.g.s. system equals the gram-calories of heat transmitted per second through a layer of substance 1 cm. thick, the area of each face being 1 sq. cm., with a difference of 1°C. between the faces. In the f.p.s. system it is the B.t.u. of heat transmitted per hour through a layer 1 in. thick with faces 1 ft. square and with a difference of 1°F. between the faces. Coefficients in the c.g.s. system can be converted to those of the f.p.s. system by multiplying the c.g.s. units by 2,900.

The amount of heat required to change the temperature of a substance is termed the "specific heat" of the substance. It is defined as the calories of heat required to raise the temperature of 1 gram of the substance 1°C. From the definition of the calorie it follows that the specific heat of water at 3.5°C. is unity. Specific heat is therefore the ratio of the quantity of heat required to raise the temperature of a substance 1° to the quantity required to raise an equal mass of water 1°.

Insulating Properties of Materials.—Table 13 shows the thermal conductivities (expressed in f.p.s. units) and the relative insulating properties of various materials. The specific heats of a number of substances are shown in Table 14 (86).

The formula that expresses the relationship of the flow of heat to the insulating properties of materials is as follows:

$$Q = \frac{k \cdot A(T_1 - T_2)}{d} \cdot t \quad (35)$$

in which Q = quantity of heat transmitted, B.t.u.

k = coefficient of conductivity, f.p.s. units.

A = area of face of wall, square feet.

d = thickness of wall, inches.

T_1, T_2 = temperatures on opposite sides of wall, degrees Fahrenheit.

t = time, hours.

TABLE 13.—THERMAL CONDUCTIVITY AND RELATIVE INSULATING PROPERTIES OF VARIOUS MATERIALS

Material	Thermal conductivity, k , B.t.u. ¹	Thickness for equal insulation, (water = 1 in.), inches	Material	Thermal conductivity, k , B.t.u. ¹	Thickness for equal insulation, (water = 1 in.), inches
Air.....	0.174	0.04	Petroleum oil.....	1.03	0.24
Snow, fresh.....	0.754	0.17	Lime.....	0.841	0.19
old.....	3.48	0.80	Portland cement.....	2.06	0.47
Water.....	4.35	1.00	Stone concrete.....	6.38	1.47
Ice, 0°C.....	14.50	3.33	Cinder concrete.....	2.35	0.54
160°C.....	19.14	4.40	Firebrick.....	0.812	0.19
Limestone.....	13.6	3.13	Glass, window.....	7.25	1.67
Quartz perpendicular to axis.....	50.2	11.55	Woods, kiln dried:		
parallel to axis.....	94.3	21.68	white pine.....	0.754	0.17
Slate perpendicular to axis.....	9.86	2.27	Virginia pine.....	0.957	0.22
parallel to axis.....	17.4	4.00	oak.....	1.45	0.33
Sandstone.....	16.0	3.68	hard maple.....	1.10	0.25
Granite.....	15.4	3.54	Sawdust.....	0.348	0.08
Soil in earth's crust, dry.....	10.7	2.46	Hair felt.....	0.240	0.06
Sand, white, dry.....	2.70	0.62	Leather, cowhide....	1.22	0.28
Diatomaceous earth....	0.377	0.09	Blotting paper.....	0.435	0.10
Mica.....	5.22	1.20	Corkboard.....	0.307	0.07
Chalk.....	6.38	1.47	Asbestos paper.....	0.493	0.11
Peat, dry.....	0.348	0.08	Paraffin.....	0.171	0.39
Silver.....	2.917	670	Vaseline.....	0.638	0.15
Copper.....	2.662	612	Rubber.....	1.71	0.25
Aluminum.....	1.392	320	Wool, packed.....	0.244	0.06
Wrought iron.....	418	96	Cotton, packed.....	0.290	0.07
Lead.....	241	55	Silk.....	0.276	0.06
Mercury.....	43	10	Linen.....	0.609	0.14

¹ B.t.u. passing per hour through a layer of material 1 sq. ft. in area and 1 in. thick, per degree Fahrenheit difference in temperature of the two faces.

According to this relationship, the time required for any given quantity of heat transfer varies directly with the thickness and inversely with the thermal conductivity of the material separating substances of different temperatures.

The total heat transfer required to change the temperature of a given mass T° equals the product of the mass in grams, the change in temperature, and the specific heat.

To illustrate, let it be required to calculate the heat loss through a firebrick wall ($k = 0.81$) 4 in. thick, 5 ft. wide, and 6 ft. high in 4 hr. when the temperature is 400°F. on one side of the wall and 160°F. on the other.

By substitution in formula (35) there results

$$Q = \frac{0.81 \times 30 \times 240}{4} \times 4 = 5832 \text{ B.t.u.}$$

From Table 14 we find the specific heat of air to be 0.2375, which equals the B.t.u. required to raise the temperature of 1 lb.

TABLE 14.—SPECIFIC HEATS OF VARIOUS SUBSTANCES

Substance	Temperature, degrees Centigrade	Specific heat
Kaolin.....	20 to 98	0.224
Quartz sand.....	20 to 98	0.191
Mica.....	20 to 98	0.2061
Granite.....	12 to 100	0.192
Sandstone.....	0.22
Limestone.....	15 to 100	0.216
Quartz.....	12 to 100	0.188
Glass, flint.....	10 to 50	0.117
Air.....	0 to 200	0.2375
Water.....	3.5	1.0000
Ice.....	-18 to -78	0.463
Asbestos.....	20 to 98	0.195
Woods.....	20	0.327
Paraffin.....	0 to 20	0.6939
Rubber.....	0.481
Asphalt cement.....	0.400

of air 1°F. To raise 1,000 cu. ft. (80.71 lb.) 1°F. would require 19.17 B.t.u. Such being the case, the 5832 B.t.u. just referred to is sufficient to raise the temperature of the air in a room 10 by 10 by 10 ft. 304°F.

If the wall were of granite ($k = 15.4$) instead of brick, the loss would be $15.4/0.81 \times 5,832 = 110,880$ B.t.u.

When the temperature of the human body is 98.6°F. , and that of the air is 10°F. , the flow of heat through cloth $\frac{1}{20}$ in. thick will

be at the rate of 1080 B.t.u. per square foot of area per hour, or 460 B.t.u. per hour depending on whether the cloth be of linen ($k = 0.61$) or silk ($k = 0.26$).

Effect of Air Temperature.—In nature heat flows from the air to the soil when the air is the warmer and from the soil to the air when the air is the cooler. When the air temperature drops below the freezing point, frost penetrates the ground. The greater the drop in air temperature and the greater the thermal conductivity of the ground the faster the penetration occurs (see Fig. 49). The greater the specific heat of the soil the slower is the penetration of the frost, the thermal conductivity remaining constant. The speed and depth of frost penetration in subgrades depend also on the heat characteristics of the pavement or other road surface and base course (92).

The influence of the heat constants (Tables 13 and 14) of both soil and road materials on the flow of heat from the ground may be easily demonstrated. Let it be assumed that after the ground surface and the air have had the same temperature (60°F.), the temperature of the air drops abruptly to 10°F. and remains there until the temperature is 32°F. in the ground at a depth of 3 ft.

The mean average temperature of the depth of crust affected is expressed by the relationship

$$T_m = \frac{T_2 + T_1}{2} \quad (36)$$

in which T_m = average temperature within the affected layer.

T_2 = temperature at the ground surface = 10°F.

T_1 = temperature of the ground 3 ft. below the surface = 32°F.

Then

$$T_m = \frac{10 \times 32}{2} = 21^{\circ}\text{F.}$$

The specific heat SH of the soil may be assumed to equal 0.20, and its weight 100 lb. per cubic foot. The quantity of heat lost per square foot of ground surface during the drop in temperature of the 3-ft. layer of soil from 60°F. to an average of 21°F. will equal

$$(60 - 21) \times 300 \times 0.20 = 2,340 \text{ B.t.u.}$$

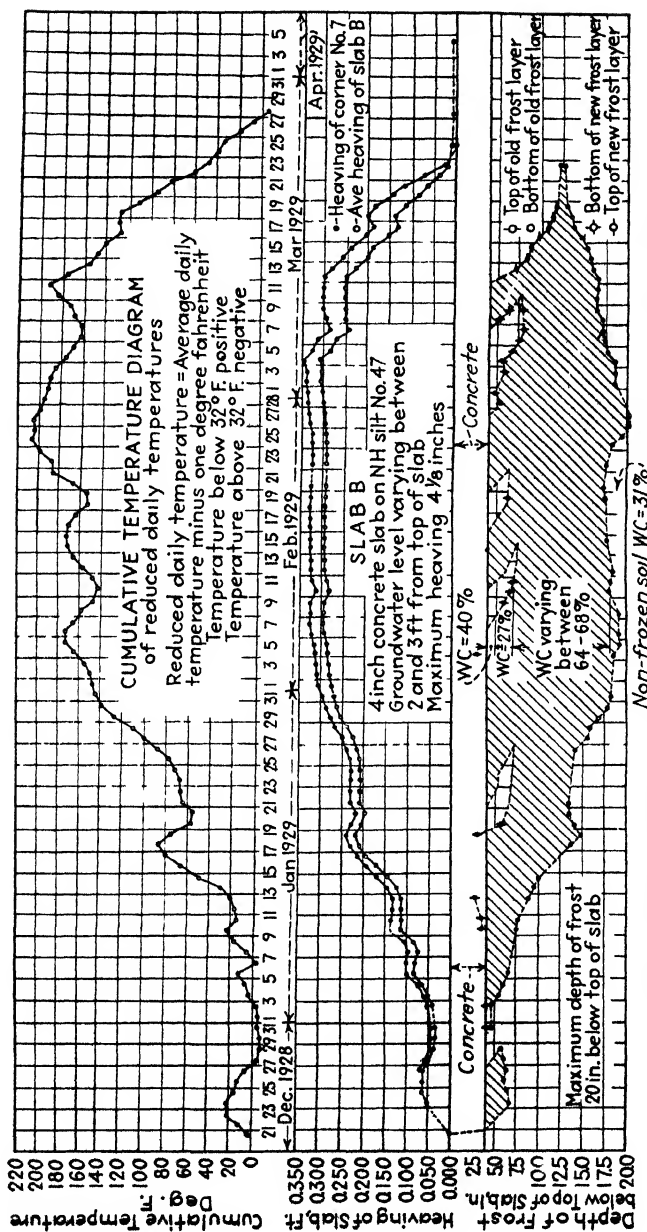


Fig. 49.—Relation between cumulative temperatures and depth of frost penetration in soil.

In the present analysis, no account is taken of any water in the soil which would release considerable heat in the process of freezing.

The temperature of the soil at any depth and at any time may be expressed by the formula

$$T = T_2 + (T_1 - T_2) 2 \int_0^{\frac{D}{h\sqrt{2t}}} \frac{e^{-\frac{x^2}{2}}}{\sqrt{2\pi}} dx \quad (37)$$

in which D = depth below surface, feet.

T = corresponding temperature, degrees Fahrenheit.

t = time in hours since a drop in air temperature from T_1 to T_2 .

h^2 = coefficient of diffusivity of the soil, feet squared per hour.

$$= \frac{k}{12 \times SH \times \text{density of soil}}$$

The integral in formula (37) is the area under the normal curve of error or probability curve. Its value for any value of the upper limit is given in tables which may be found in various handbooks. If D is 3 ft. and k is 5.0 and the other factors are as before, equation (37) gives the temperature of the soil as 58°F. after 48 hr., 53°F. after 96 hr., 44°F. after 216 hr., and 33°F. after 600 hr. or 25 days.

Effect of Types of Covering.—Let it now be assumed that a slab of concrete ($k = 6.38$) 6 in. thick is placed on the surface of the ground and that a cubic foot of the soil contains 20 lb. of water which is to be frozen by the conduction of heat through the slab to the air. If the temperature of the top foot of soil is assumed to be constant at 32°F. and the air remains at 10°F., the time required to transfer the heat given off in freezing may be computed. Since the heat of fusion of water or the amount of heat required to melt 1 lb. of ice is 144 B.t.u., the amount of heat to be conducted through the slab is $20 \times 144 = 2,880$ B.t.u. per square foot. The time required for the transfer of the 2,880 B.t.u. is

$$t = \frac{2,880 \times 6}{6.38 \times (32 - 10)} = 123 \text{ hr.}$$

For coverings of other materials with like thickness of 6 in. the times required for the heat transfer would be as follows:

Dry sand ($k = 2.70$), 291 hr.

Virginia-pine plank ($k = 0.957$), 821 hr.

Fresh snow ($k = 0.754$), 1,040 hr.

Dry peat ($k = 0.348$), 2,260 hr.

The relative depth of frost penetration for different types of coverings is largely dependent upon their thermal conductivities. Let it be assumed that during a period of low temperature, enough heat escapes through a covering of concrete to cause frost to penetrate to a depth of 2 ft. During an equal period of time enough heat could flow through coverings of other materials having the same thickness as the concrete to cause the frost to penetrate to depths as follows:

$$\text{Wrought iron, } \frac{24}{6.38} \times 418 = 1,572 \text{ in.} = 131 \text{ ft.}$$

$$\text{Dry soil, } 3.76 \times 10.7 = 40.23 \text{ in.}$$

$$\text{Dry sand, } 3.76 \times 2.70 = 10.15 \text{ in.}$$

$$\text{Pine planks, } 3.76 \times 0.96 = 3.61 \text{ in.}$$

$$\text{Fresh snow, } 3.76 \times 0.75 = 2.82 \text{ in.}$$

$$\text{Dry peat, } 3.76 \times 0.35 = 1.32 \text{ in.}$$

Failure of the heat to flow upward through the soil as rapidly as it can escape through the covering on top would, of course, prevent penetration anything like that indicated on iron.

Except for the iron, however, these figures do not exaggerate the variations in the depth of frost penetration found at different locations under the same climatic conditions. The depth of frost under a road surface has been observed to be as much as 4 ft. when at the same time the depth in adjacent ground, covered with humus, leaves, and possibly snow, is less than 1 ft. Under the road surface the depth is likely to be less before the removal of snow than afterward. Also, the depth of frost under the center of the pavement where the snow packs the soonest is usually greater than along the edges adjacent to where the snow is wasted on the shoulders. Owing to the high conductivity of iron and steel, the greatest depth of frost may be expected to be found along the sides of or beneath fire hydrants, iron posts, and the like. For the same reason it is possible for frost to occur along pipe lines

located below the depth of frost in the adjacent soil. Were it not for the fact that the soil is in contact with relatively small cross-sectional areas of iron, serious trouble could be expected in such cases due to the rapid loss of heat and resulting frost heave.

Further drop in the temperature of the air after frost has penetrated the ground causes the frozen layer to increase in thickness. Rise in the temperature of the air up to the freezing point causes thaw to begin at the bottom of the frozen layer and proceed

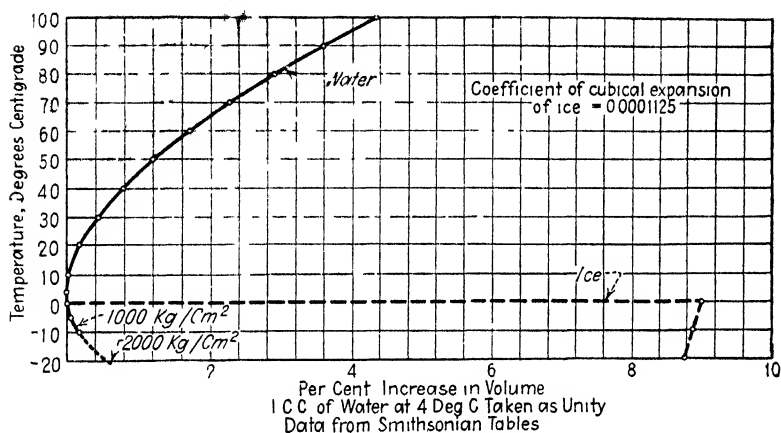


FIG. 50 — Relation between temperature and volume of water

upward. When the air temperature rises above the freezing point, thaw proceeds both from the surface of the ground downward and from the bottom of the frozen layer upward.

Growth on Freezing.—As a rule, materials in nature contract with lowering temperature and expand with rising temperature. Water is a notable exception. It contracts with lowering temperatures only until 4°C. is reached. With further drop in temperature, water expands (see Fig. 50).

Accordingly, 104.34 cu. ft. of water at 100°C. contracts with lowering temperature to a volume of 100.00 cu. ft. at 4°C. On further reduction in temperature the volume increases, becoming 100.01 cu. ft. at the normal freezing point, of 0°C. If the water is kept from freezing below 0°C. by means of pressure, the continued expansion would cause the volume to increase to 100.19 at -10°C. Without change of pressure the freezing point of water may be

reduced by the presence of chemicals in solution, as illustrated in Table 15 (93).

Independent of the contraction and expansion of water due to change in temperature, its volume increases by 9 per cent, with change from the liquid to the solid state. As a result the 100.01 cu. ft. of water referred to above, if frozen at 0°C., would become 109.01 cu. ft. of ice.

TABLE 15.—FREEZING TEMPERATURES OF CALCIUM CHLORIDE AND COMMON-SALT BRINES AT VARIOUS CONCENTRATIONS

Concentration, per cent	Freezing temperature, degrees Fahrenheit	
	Calcium chloride brine	Sodium chloride brine
5	29.0	+27.0
10	23.0	+20.4
15	12.5	+12.0
20	- 3.0	+ 1.8
21	- 5.5	- 0.8
22	-10.5	- 3.0
23	-15.5	- 6.0
24	-20.5	+ 3.8
25	-25.0	+16.1

In bodies of free water the water that expands with drop in temperature below 4°C. rises to the surface just before freezing. The additional growth on freezing further decreases its density and makes doubly certain that the ice thus formed will float on the surface.

If, instead of floating, ice should sink, shallow bodies of water would completely solidify from the bottom upward, instead of freezing only at the surface. This would contribute to the gradual extinction of fish or other animals in cold climates, which depend upon water furnished by small streams and lakes for existence.

Direction of Growth.—The growth of water on freezing occurs principally in the direction of heat loss. If a glass cylinder is filled with water and so exposed to freezing temperatures that the loss of heat occurs in all directions, the growth on solidification will likewise occur in all directions and consequently break the cylinder. If, however, the cylinder or bottle is insulated on the

sides so that heat will be lost only through the top, the freezing will begin at the top and proceed downward, with the growth's occurring only in a vertical direction. In this manner the water may be frozen solid without bursting the glass container.

Fortunately, the direction of heat loss and the resulting growth of natural bodies of water on freezing also is mostly vertical. If the growth were equal in all directions, like volume changes due to contraction and expansion, it would exceed 2 ft. per hundred, transversely, longitudinally, and vertically. The resulting damage to locks, dams, river banks, bridges, and all other similar structures as well as buildings and the like located in freezing ground can easily be visualized especially when one considers that ice may exert a horizontal thrust exceeding 50,000 lb. per square foot.

While the growth of water on freezing is necessary for the preservation of animal life, the primarily vertical direction of its occurrence serves to minimize its detrimental effects to the extent that rivers, canals, and lakes can be frozen repeatedly without damaging contiguous structures by lateral thrust.

What seem to be integral and homogeneous sheets of ice on natural bodies of water are actually myriads of vertically elongated ice needles held in close contact by a strong horizontal bond. The "honeycomb" structure which often occurs during the thaw of natural ice clearly discloses these needles.

It should not be inferred from the foregoing that ice never exerts a lateral thrust. Such thrust occurs but is due to causes other than the simple growth of water on freezing.

Figure 51 shows a bank of soil composed of sod, boulders, and clay which was thrown up by ice thrust on the edge of a lake in Wisconsin. On the average this bank was 4 ft. high, 11 ft. wide at the base, and 4 ft. wide at the top. At one place it was at least 8 ft. high and had raised a tree of considerable size through this distance. In many places boulders were rammed into the bank in such manner that they presented much the same appearance as plums in a pudding. The bank was raised up and turned over by the ice shove, sometimes dislodging and moving trees up to 12 in. in diameter.

The cause of ice ramparts has been described by Gilbert (94) as follows:

The ice on the surface of a lake expands while forming so as to crowd its edge upon the shore. A further lowering of temperature produces contraction, and this ordinarily results in opening vertical fissures. These admit water from below, and by the freezing of that water they are filled, so that when expansion follows a subsequent rise of temperature, the ice cannot resume its original position. It consequently increases its total area and exerts a second thrust upon the shore. When



FIG. 51.—Result of ice thrusts on Picnic Point near Madison, Wisconsin (Photograph by Dr. E. R. Buckley, Courtesy of Wisconsin Academy of Sciences, Arts, and Letters.)

the shore is abrupt, the ice itself yields, either by crushing at the margins or by the formation of anticlines elsewhere; but if the shore is generally shelving, the margin of the ice is forced up the acclivity and carries with it any boulders or loose material about which it may have frozen. A second lowering of the temperature does not withdraw the protruded ice margin but initiates other cracks and leads to a repetition of the shoreward thrust. The process is repeated from time to time during the winter but ceases with the melting of the ice in the spring.

Ice Lenses Responsible for Heave.—The growth of soil moisture on freezing and the resulting volume increase in soils also occur principally in a vertical direction. Consequently, such deformations are rightly termed "heave." However, detrimental frost heave involves more than the mere expansion of water due to drop in temperature and the well-understood attendant growth due to solidification. In a thoroughly saturated soil with a pore

ratio of 0.8, the maximum heave due to the 9 per cent increase in the volume of the contained water on freezing would be only 0.48, 1.2, and 2.4 in. for respective depths of frost penetration of 1, $2\frac{1}{2}$, and 5 ft.

Instead, detrimental heave for these depths of frost penetration may range from several inches to several feet and may involve either an explosive action of solidifying water or an accumulation of excess moisture in the soil.

If, for example, the soil just referred to as having a pore ratio of 0.8 would, upon freezing to a depth of 5 ft., heave 2 ft., or 24 in., it can safely be assumed that 21.6 in. of the heave—the excess over 2.4 in.—would be due to an accumulation of new ice.

The new ice occurs as clear horizontal lenses distributed throughout the depth of frost penetration. Their aggregate thickness would account for all of the 21.6 in. excess heave. The layer of water 19.8 in. thick ($21.6 \div 1.09$) required for the segregated lenses would have been drawn up into the frozen layer during its formation. It is quite likely, however, that the pore ratio (0.8) of the soil between the ice lenses would remain the same as that of the soil prior to being frozen.

It has been suggested by J. L. Harrison and I. B. Mullis (95) that the explosive action of frost is due to the instantaneous freezing of supercooled water when the pressure required for supercooling is released. Two theories have been advanced in explanation of the accumulation of additional water required to form the ice lenses. Benkelman and Olmstead (96) suggested that the formation of such lenses is due to the increase in volume of free or bulk water on freezing and the corresponding decrease, upon thawing, in combination with alternate freezing and thawing. Taber (97) and Bouyoucos (98, 99, 100), working independently, ascribed the segregation of moisture upon freezing to phenomena involving the physics of capillary and film moisture.

Freezing of Supercooled Water.—If when water is cooled below 4°C . its expansion is prevented, high pressures are developed, the effect of which is first to reduce and, on further increase in pressure, to raise the freezing point, as noted previously. According to Fig. 34 (Chap. V), supercooling to about 14°F . causes water to exert a pressure greater than 1,000 tons per square foot.

When the pressure exerted by the supercooled water breaks the container, the supercooled water is released and instan-

taneously turns into ice with an expansion of 9 per cent in volume. This is similar to the instantaneous changing to steam of all superheated water when a boiler explodes.

This explosive action of ice is illustrated by experiments by Major Williams (101) and described as follows:

Having quite filled a 13-in. iron bombshell with water he firmly closed the touchhole with an iron plug weighing 3 lb. and exposed it in this state to the frost. After some time the iron plug was forced out with a loud explosion and thrown to a distance of 415 ft., and a cylinder of ice 8 in. long issued from the opening. In another case the shell burst before the plug was driven out, and in this case a sheet of ice spread out all round the crack. It is probable that under the great pressure some

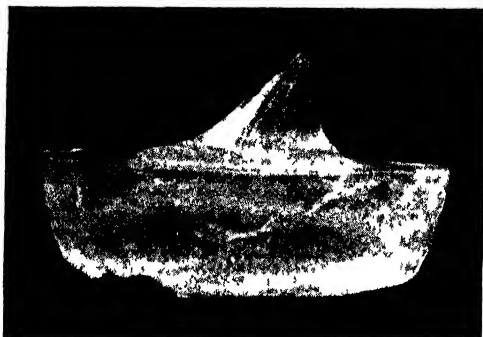


FIG. 52.—Cake of ice which froze on all sides and burst under expansive force of freezing water in the interior.

of the water still remained liquid up to the time at which the resistance was overcome; that it then issued from the shell in a liquid state, but at a temperature below 0°C. (32°F.), and therefore instantly began to solidify when the pressure was removed and thus retained the shape of the orifice whence it issued.

The instantaneous solidification of supercooled water is illustrated in Fig. 52. This occurrence is explained as follows: The water that formed this cake was partly frozen in an aluminum pan. Owing to the high degree of heat conductivity of the aluminum (see Table 13), freezing probably began simultaneously at the top, bottom, and side of the water mass, thus enclosing the unfrozen water in an ice container. As the process of freezing continued, the contained water exerted increasing pressure until the resistance of the ice container was exceeded. The pressure being suddenly released, the supercooled water immediately

solidified, the 9 per cent expansion due to freezing causing the protruding tooth to be formed, as shown on the top of the cake.

It can be easily seen that after the surface of frozen soil has thawed out and then refreezing begins, water trapped between

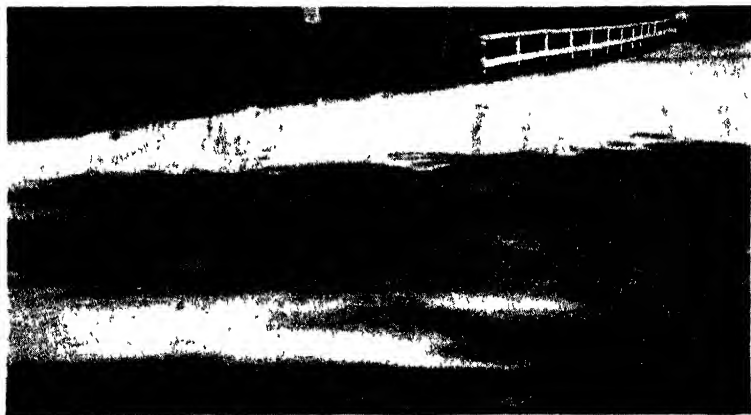


FIG. 53.—Frost heave in a surface-treated macadam road.

the frozen undersoil and newly frozen surface soil might produce explosive action similar to that described. Figure 53 is a photograph of a heaved pavement which may be the result of just such an occurrence.

Figure 54 illustrates the occurrence of soil blisters which, according to Nikiforoff (102), may extend more than 20 ft. in

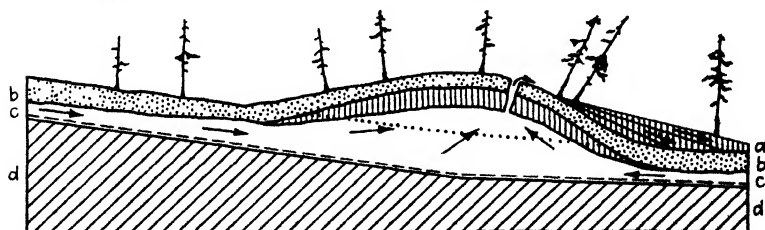


FIG. 54.—Cross section of a soil blister in Siberia. (Courtesy of Soil Science.)

height in Siberia. The method of formation is explained as follows: *d* represents ever frozen subsoil which extends to considerable depths. The soil above this stratum thaws during the summer and, as is characteristic of most arctic regions, is frequently quite wet, especially near the foot of slopes and in surface depressions where water collects. At the beginning of winter the surface

soil *b* freezes down to the water in liquid form *c*. At this stage the liquid water is confined between two zones of hard frozen material; when the pressure becomes sufficiently high to rupture



FIG 55 —Ice-covered slope of a soil blister (Courtesy of Soil Science)

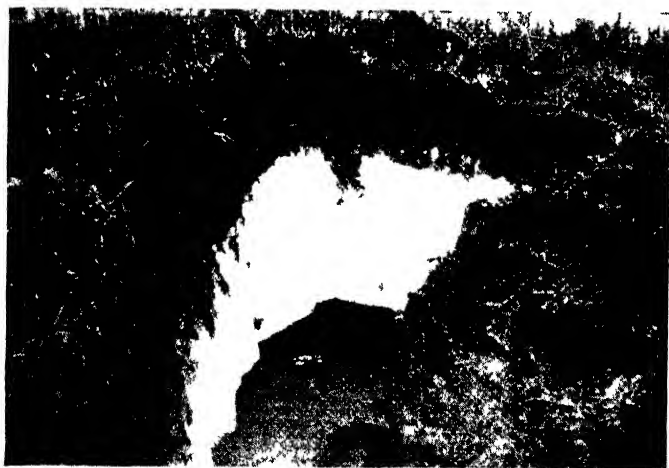


FIG 56.—Ice cavern formed in soil blister (Courtesy of Soil Science)

the frozen surface soil, the supercooled water is forced through the crack and is probably frozen instantly. Figure 55 shows the ice-covered slope of a soil blister. The cavern formed by the heave is shown in Fig. 56. These may attain a height of 6 and a width of 30 ft.

Heave in Permeable Soils.—The formation of ice layers in completely saturated permeable soils may be accounted for by the fluctuations in the depth of the frost line which occur due to change in the air temperature as long as the temperature of the air remains below the freezing point.

With reference to Fig. 57, *A* represents a column of unfrozen soil. With a drop in temperature the frost line penetrates as shown in *B*, and the soil freezes, resulting in a heave equal to

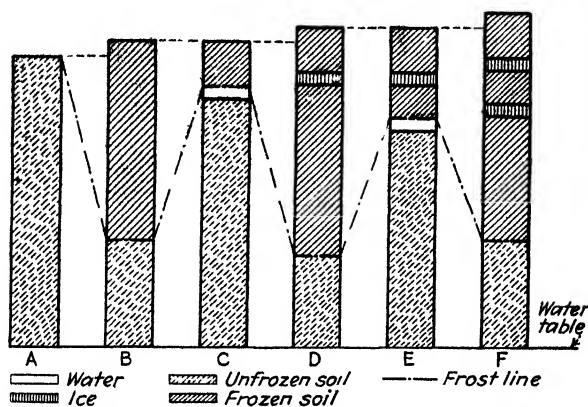


FIG. 57.—Ice layers formed by fluctuations in frost line.

9 per cent of its moisture thickness. On rising temperature the under portion of the frozen layer melts and returns to normal volume, thus leaving a crevice under the frozen layer equal in thickness to 9 per cent of the original thickness of the water in the portion thawed. This crevice then fills with water as shown in *C*, and on the next temperature drop *D* becomes an ice layer. A repetition of this process produces a second crevice and ice layer as shown in *E* and *F*.

Heave in Soils of High Capillarity.—The growth of ice layers in the fine-grained soils (Figs. 58, 59, and 60) depends upon three physical facts:

1. Particles of capillary water contained in the larger soil pores freeze at either normal or slightly less than normal temperature, say, -1 to -4°C .
2. Particles of cohesive water contained in much smaller soil pores resist freezing at temperatures considerably lower than -4°C .
3. During the process of freezing, particles of capillary water attract from adjacent fine capillaries the films of cohesive water which individually do not freeze.

When drawn into contact with existing ice crystals, however, the cohesive moisture changes to the capillary variety and freezes and thus increases the size of the existing crystals. Continuation of this process causes the ice crystals to increase in thickness as

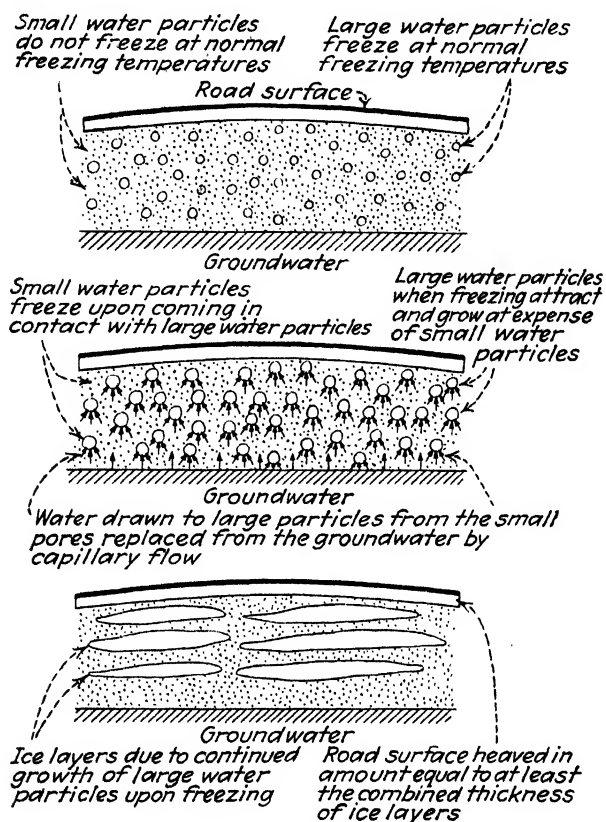


FIG. 58.—Physics of frost heave.

long as they are being supplied with either capillary or cohesive moisture through the fine capillaries from the ground-water supply below. The force with which the frozen particles attract the unfrozen ones is, according to Taber (97), sufficient to lift a column of water almost 500 ft. in length.

Water that forms crystals, however, may be drawn from any adjacent pocket as well as from the ground water. Figure 61 illustrates the occurrence of frost crystals and thin ice sheets

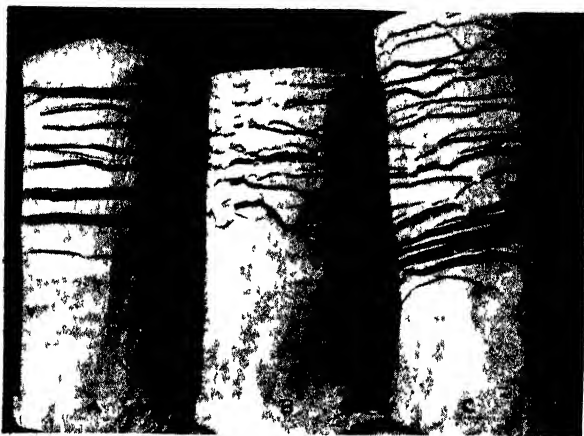


FIG. 59.—Ice lenses in frozen-soil cylinders prepared by Prof Stephen Taber



FIG. 60.—Ice lenses in soil from highway subgrade in Minnesota.

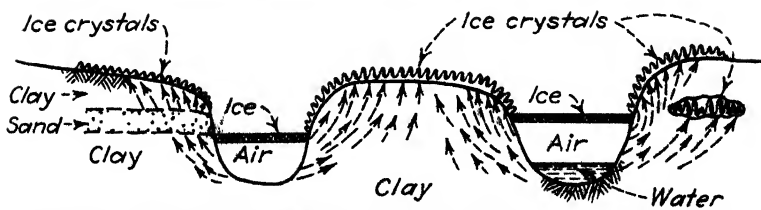


FIG 61 — Water movements and ice formation on a rutted wet clay surface.

frequently observed on clay roads that are more or less rutted. When the temperature of the ground surface reaches the freezing point, ice crystals form on the surface of moist soil, and ice forms on the surface of any pools of water that may exist on the surface. Following these freezing temperatures one frequently notices that these recently water-filled depressions are covered with thin sheets of ice with little or no water underneath. Upon further examination it will be noted that all fissures or other soil openings contain ice crystals. The absence of water underneath ice-covered depressions is due to the water's having been withdrawn from under the ice sheets by the increased capillary tension of the soil produced by lowered temperature. It will be noted that ice crystals on the surface of clay soils will be longer than on those soils which contain less clay. Compacted clay soils will, however, contain fewer frost crystals at the surface than a similar soil in a less compacted state.

Moisture Requirements for Frost Heave.—Whether or not frost heave occurs depends upon the quantity of water available for freezing at a given location. This moisture may be furnished by complete saturation in undrained sandy soils or by capillarity in the silts. In the silts neither the height to which water will rise by capillarity eventually nor the rate of such rise is alone the determining factor, but, instead, the quantity of moisture capable of being raised to a given height above the water table in a given time. Just as the amount of water furnished by a pipe depends upon the pressure acting on the water, the diameter of the pipe, and the frictional resistance to flow, the raising of a quantity of water sufficient to produce frost heave in the subgrade at a given height above the ground-water elevation (the rate at which capillary equilibrium is destroyed being disregarded) depends upon the force of capillarity, the area of pore space, and the frictional resistance.

Capillary pressure varies inversely with the diameter of the pores. The frictional resistance to flow through a soil is a function of the surface area of the soil particles and consequently increases with a decrease in grain size at a much greater rate than does the capillary pressure. Therefore, in order to furnish capillary moisture in detrimental amounts the pore size must be small enough to furnish appreciable capillary pressure but large enough to prevent too much frictional resistance to flow.

Effect of Soil Character.—In cohesionless, well-drained sands possessing but little capillarity, no detrimental frost heaves are likely to occur, because practically all of the contained water freezes at normal freezing temperatures, and cohesive water particles do not exist in amounts sufficient to cause appreciable segregation. Silt soils, however, with relatively high capillarity and relatively low cohesion are likely to heave in very appreciable amounts.

The moisture films in cohesive soils travel very slowly, owing to capillarity. Consequently, in dense clays with low ground-water level and absence of lateral seepage, only limited amounts of water are available for ice segregation. Therefore, considerable heave is not likely to occur.

The effect of size of grain on the speed of capillary rise, indicated by Table 16 (22), indicates that particles with sizes ranging from 0.1 to 0.01 mm. should be the most productive of frost heave.

TABLE 16.—RISE OF WATER IN GRAINS OF DIFFERENT SIZES

Diameter of grain, millimeters	Rise in 24 hr., inches	Maximum rise, inches	Period required to attain maxi- mum rise, days
5 to 2	0.87	1.0	3
2 to 1	2.13	2.6	4
1 to 0.5	4.53	5.2	4
0.5 to 0.2	8.43	9.7	8
0.2 to 0.1	14.80	16.9	8
0.1 to 0.05	20.87	41.5	72
0.05 to 0.02	45.39	78 ±	53 +
0.02 to 0.01	19.09		
0.01 to 0.005	11.22		
0.005 to 0.002	5.63		
0.002 to 0.001	2.17		

A detailed analysis of the cause of 156 frost heaves examined in Michigan by Burton and Benkelman (103) showed that 94 occurred in layers or pockets of fine-textured material such as silt, very fine sand and silt, or silty clay, clay, or sandy clay surrounded by coarser, better drained material; 22 occurred in pockets of silt or of very fine sand and silt surrounded by clay; 30 in medium to fairly coarse sand; and 6 in moderately coarse textures. The remaining 4 of the 156 heaves occurred where the

subgrade soil appeared uniform with respect to character and moisture content.

Of the 94 heaves that occurred in finely divided soils, 5.3 per cent developed in silt, 32 per cent in very fine sand and silt, 4.3 per cent in very fine sand, 30 per cent in clay, 8 per cent in silty clay, and about 17 per cent in sandy clay. Of the 22 heaves that occurred in finely divided textures surrounded by clay, 45 per cent developed in silt, 23 per cent in very fine sand and silt, 18 per cent in silty clay, and 14 per cent in sandy clay.

The heave was somewhat less for sandy clays or loams than for clays. The average height for all the heaves occurring in silt was 6 in., in very fine sand and silt 5 in., in very fine sand 4 in., in clay 4 in., in silty clay 5 in., and in sandy clay 3 in.

Of the heaves studied, 80 per cent occurred in cuts, 2 per cent on fills, and 18 per cent on locations extending from cut to fill. Of 500 heaves located in a preliminary survey, 76 per cent occurred in cuts, 10 per cent on fills, and 14 per cent on cut-to-fill locations. Of 141 heaves in cuts, 80 per cent were located at a depth of 4 ft. or more below the original ground elevation.

In a later report on Michigan conditions Benkelman and Olmstead (96) state that examinations of the subgrade soil at about 200 locations in the northern section of the state revealed ample evidence that heaves in excess of a few inches may occur in coarse sands or even in gravelly materials provided that an excess of water is present. It was found that approximately 70 per cent of the heaves occurred in sandy materials of decidedly low capillarity in which, however, a large excess of free water was present, either from seepage over impervious strata or from a naturally high water table. It must be understood that in more "open" materials a supply of free water must always be available to fill void spaces created by upward thawing, because if an opportunity for drainage prevails, these soils will not ordinarily contain sufficient moisture by capillarity to satisfy the requirements for heaving according to the stated theory. In silts and very fine sands, however, even though drainage is provided, the amount of moisture either present by capillarity or drawn to the zone of freezing will be sufficient to fill voids created during a cycle of isothermal line fluctuations and will cause excessive heaving.

It is an accepted fact that capillary water cannot be removed by artificial drainage and, therefore, that those textures which are

capable of supplying water in sufficient amount by capillarity for heaving to occur by virtue of frost line fluctuations must be removed from the grade and replaced with materials having favorable drainage characteristics. On the other hand, elimination of frost heaving in coarser materials involves only the installation of properly designed tile drainage systems in order to remove gravitational water which may enter the soil by seepage over impervious strata or which may exist from a naturally high ground-water elevation.

Results of Controlled Experiments.—The difference in the heaving properties of sands and silts is illustrated excellently by Casagrande's (92) observations made daily at the Massachusetts Institute of Technology from Dec. 24, 1928, to Mar. 28, 1929, of the movement of various types of pavement slabs 3 ft. square resting on different kinds of soil contained in specially constructed boxes in which the water supply was regulated. He found that the maximum heave of slabs laid on silt varied from 2.3 to 3.8 in., whereas the heave of similar slabs laid on fine sand was but from 0.02 to 0.13 in.



FIG 62.—Frozen cylinder, half sand and half clay, much segregated ice in clay but not in sand. (Photograph by Prof. Stephen Taber.)

The contrast between the heaving properties of sand and clay is illustrated in Fig. 62.

In Fig. 63 is shown the heaving record and soil conditions along 2,000 ft. of concrete pavement in New Hampshire taken during the winter of 1927–1928.

The study of the data secured at the Massachusetts Institute of Technology and of similar information from many sections of New Hampshire lead to the conclusion that

Under natural freezing conditions and with sufficient water supply one should expect considerable ice segregation in nonuniform soils containing

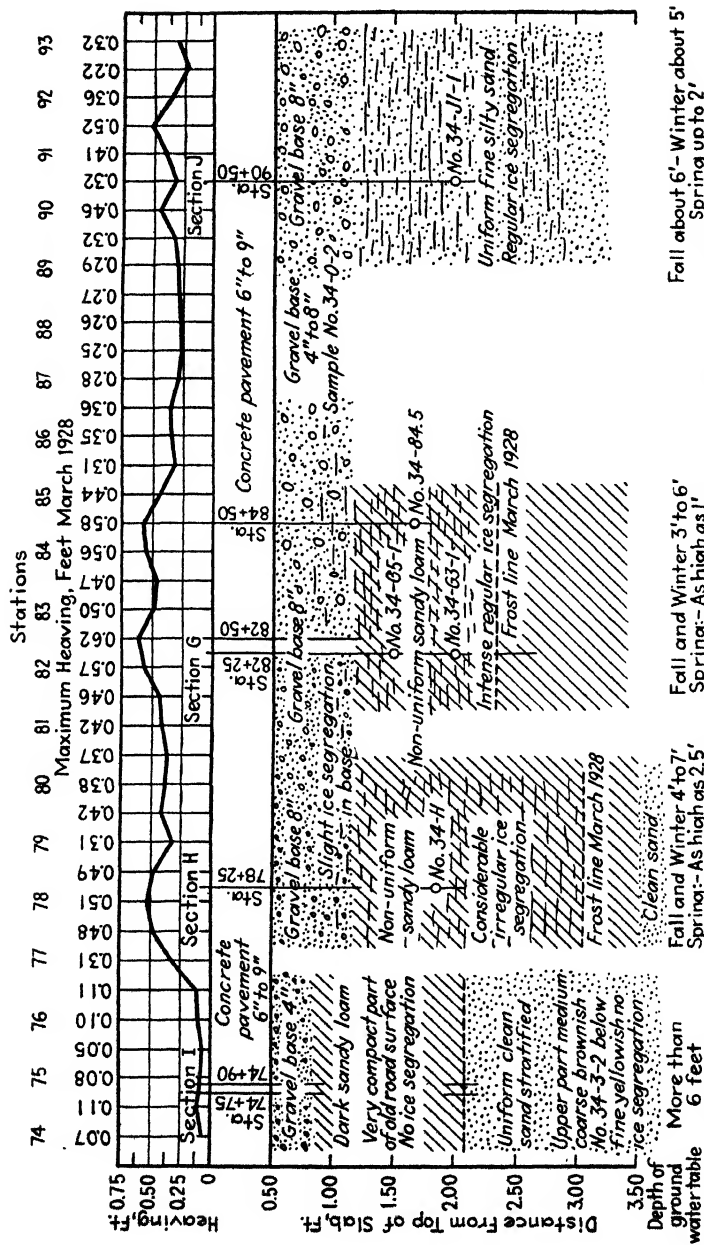


Fig. 63.—Record of frost heave and soil conditions along concrete pavement section in New Hampshire.

more than 3 per cent of grains smaller than 0.02 mm. and in very uniform soils containing more than 10 per cent smaller than 0.02 mm. No ice segregation was observed in soils containing less than 1 per cent of grains smaller than 0.02 mm., even if the ground-water level was as high as the frost line.

Effect of Climatic Changes.—According to observations in Minnesota (103) after the exceedingly dry summer of 1930, a sudden drop in temperature following several days of heavy rainfall froze the subgrade on highways in Minnesota to a depth of about 2 ft. without producing any appreciable amount of heaving. However, a subsequent thaw which drove practically all of the frost from the subgrade was followed by a period of relatively slow frost penetration interspersed with more rapid fluctuations. During this period there developed what was probably the most severe case of frost heaving experienced on the highways of this state. This suggested that the first freezing and subsequent thawing acted as a primer in saturating the comparatively dry upper subgrade and establishing maximum capillarity for the second freezing period. Similarly, observations made on slabs at Fort Collins, Colo. (Agricultural College), laid on dry soil with exceptionally low water table showed the slabs to settle instead of heaving with freezing temperatures.

Effect of Soil Profile.—In many sections of the country a road surface rarely rests on uniform subgrade material for any great length. This is due to two reasons:

1. The grade line will traverse one or more of the soil layers of a soil profile depending on the depth of cut, and each layer, possessing certain definite physical characteristics, will form a portion of the subgrade.

2. An individual soil layer may possess such variable characteristics as

- a. Pockets of soil material differing greatly in properties from those possessed by the remainder of the layer.

- b. Stratifications within the layer.

- c. Depressions in the profile of the layer boundaries which act as reservoirs for the collection of water in excessive amounts.

- d. Variation in the ground-water elevation due to the topography of the adjacent area.

Differential heaving of pavements productive of dangerous traffic hazards is generally due to variations in the soil profile, especially with respect to their water-furnishing ability.

The variation in percolation rates of the various soil layers or soil material causes and controls seepage or lateral water

movement. Seepage develops when the amount of water percolating through any one soil layer exceeds the percolating rate of that below. Thus, water movement depends not only on the character of any particular layer but also on its relation to other associated layers. A silt loam may block or retard percolation in a soil profile where the silt is overlain by sands or may act as a water carrier if underlain by compacted sands or clays. In some soil profiles clay layers may act as water carriers if underlain by denser, less pervious layers. Stratified soils, dense clays, and hardpans are frequently causes of seepage. Silts, sands, and gravels are the most prolific carriers. Cleavage planes in geological materials are also channels for lateral water movement.

In New Hampshire, for example, frost heaves were found to be associated mainly with stratified silty clay loams and very fine sandy loams which retarded percolation or had seepage characteristics and carried the water into the subgrade, where it froze. In Minnesota the severe frost heaves are found to occur most frequently in the extremely variable glaciated soil materials of this region. Less severe heaving occurs in the structureless loessial soil material of southeastern Minnesota.

Figure 64A shows the relation between a frost heave and the soil profile of a loessial soil in southeastern Minnesota. The frost heave is confined to that portion of the road resting on the unweathered or slightly weathered structureless silt. The weathered upper layers of the profile are granular in structure and apparently do not suffer detrimental frost heave.

Figure 64B is an example of frost heave on a surface-treated gravel road in New Hampshire. The portion of the pavement located directly over a pocket of stratified sand and silt heaved as much as 10 in. above the adjoining sections. The photograph (Fig. 65) shows the condition of this road after frost had left the ground.

A classification of frost heaves occurring in different soil types in Michigan shows that these disturbances occur according to geological origin in the following order:

	Per Cent
Moraine.....	65
Shallow outwash.....	15
Till plain.....	12
Lake bed.....	4
Deep outwash.....	4

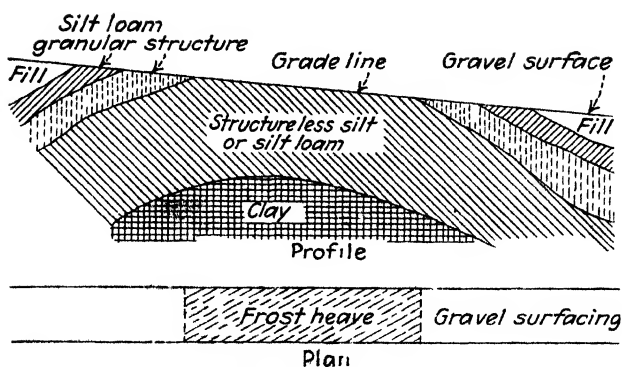


FIG. 64A.—Frost heave in loessial soil.

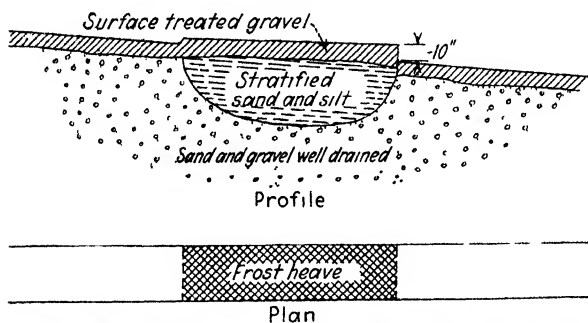


FIG. 64B.—Frost heave in a pocket of stratified sand and silt.



FIG. 65.—Bituminous surface damaged by frost action.

It is quite apparent that by far the largest number of frost disturbances are located in morainic soils with the next most frequent occurrence in shallow outwash.



FIG. 66.—Heaved gravel road at beginning of thawing period



FIG. 67.—Prominent heave in a gravel-road surface.

Effect on Road Surfaces.—Frost action affects road surfaces differently, depending on the type of surfacing. Rigid pavements heave and crack in excessive amounts but usually settle back in place after the frost leaves the ground. Successive heaves, however, are often sufficient to break up long

slabs of the thinner pavements to such an extent that replacement is required. In semirigid pavements the cracking and breakage due to one period of frost may require considerable patching, if

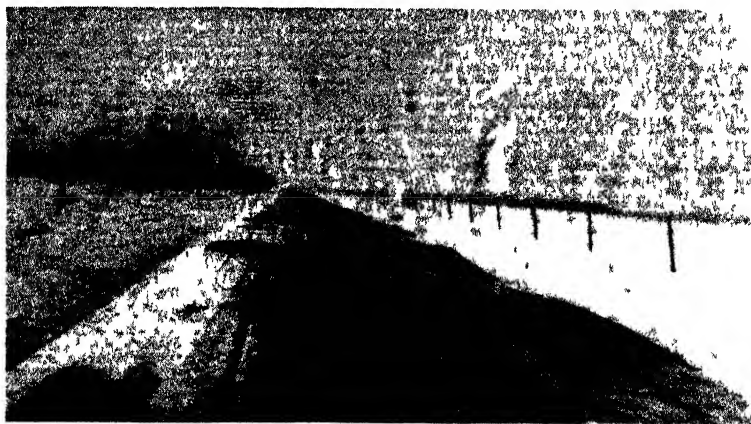


FIG. 68 —A series of heaves in a gravel-road surface.



FIG. 69 —Effect of heaving on a gravel road.

not replacement. Low-type wearing courses, such as gravel, are often entirely lost after one season, the gravel mixing with the soft undersoil without furnishing stability.

Figures 66, 67, 68, and 69 are examples of damage resulting from frost action.

When heave occurs abruptly, it constitutes a hazard to traffic. It is not uncommon for short sections of pavement to heave 10 or 12 in. above the adjacent surface, thus becoming an obstruction capable of throwing fast-moving vehicles off the road. An example of such heaving is shown in Fig. 70.

When thaw occurs, each space where an ice lens has been becomes filled with water 9 per cent less in volume than the ice lens. In addition, the new water drawn into the frozen layer during its formation is released to mix with and make the soil much wetter than before. Burton and Benkelman (103) found

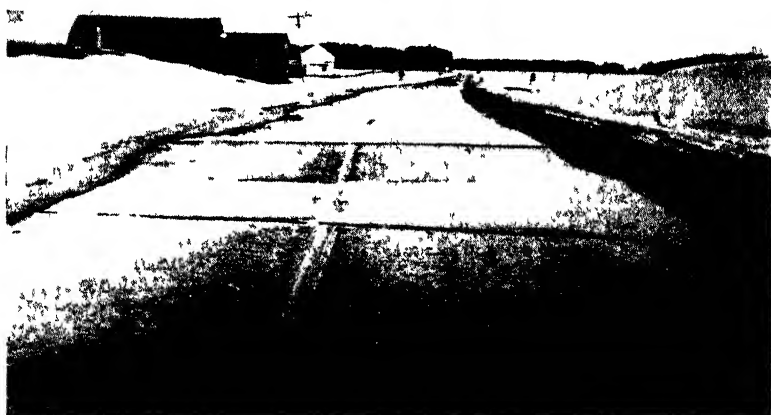


Fig. 70.—Abrupt and hazardous frost heave of a concrete pavement.

that the moisture content of a silty clay soil after freezing in the laboratory was 82 per cent, representing a gain of about 50 per cent over that present prior to freezing.

During thaws rigid and semirigid pavements may adjust themselves so that traffic can be carried without resorting to any special construction. Traffic on gravel roads, however, expedites the mixing of the liberated water with the subgrade soil, producing a soupy mud with no semblance of stability.

When the air temperature remains just slightly below the freezing point the soupy soil forms underneath the frozen layer, which gradually diminishes in thickness. When it is finally too thin to bear traffic it breaks in spots through which the soft undersoil extrudes to form what are termed "frost boils."

In the cases where the greatest depth of freezing is at the center of the road, thaw is likely to proceed fastest there also. This

produces an especially detrimental condition because the water thus liberated is prevented from draining away by the soil which yet remains frozen beneath and beside the thawed area.

A study of the damage to state highways during the winter of 1927-1928 was made by the *Engineering News-Record* (104), and approximate estimates of frost damage were submitted by various state-highway organizations. Minnesota reported 2,000 miles of highway affected by ground freezing and stated that the loss by reason of interrupted traffic and delayed business would be still greater than the direct damage. Michigan reported an average frost damage of \$50 per mile on gravel, \$100 per mile on macadam, and \$150 per mile on concrete pavements. Ohio estimated frost damage at one-quarter million dollars. Maine reported an estimated cost of half a million for required repairs. Lacking complete reports from all states but taking the reports available and supplying an approximate estimate where state-highway reports are not available, the amount of state-highway repairs required, covering frost damage during the winter season 1927-1928, is approximately three million dollars.

Relatively short sections of gravel roads often become impassable to any type of traffic during thaws. If sufficient in number, the impassable sections render the entire highway unsuitable for traffic for a period of several weeks during the spring and thus tend to isolate the towns located along the highway. Those who dare to venture forth usually require assistance in getting out of the mud before they get very far.

Therefore, aside from the large maintenance costs which arise from the failures mentioned above, there is a heavy economic loss, intangible in value, inflicted on the users of these highways.

Problems

1. Define (a) calorie, (b) British thermal unit, (c) thermal conductivity, (d) specific heat.
2. Referring to Table 13, compute the thermal conductivity in c.g.s. units of water, limestone, soil in earth's crust, dry peat, and diatomaceous earth.
3. What would be the heat loss per square foot of surface through a layer of peat 12 in. thick in 12 hr. when the temperature of one face is 10°F. and that of the other face is 40°F.?
4. The temperature of a sandy soil ($k = 2.70$; specific heat, 0.20; weight, 100 lb. per cubic foot) is 32°F., which is also the air temperature. The air temperature suddenly drops to 2°F. and remains there for a period of 10 days. What is the depth of frost penetration at the end of the 10-day period?

5. In what direction does soil moisture grow on freezing?
6. How may supercooling of water contained in a soil take place, and what may result when the supercooled water freezes?
7. Discuss frost heave in permeable soils; in capillary soils.
8. What factors influence the extent of heaving in soils?
9. Discuss how different road surfaces may be affected by heaving of the subgrade soil due to freezing.

PART III
STRUCTURAL PROPERTIES OF SOIL

CHAPTER VIII

THE PRESSURE BULB

Investigations of the structural properties of soil differ, depending on whether it is located within or below the zone of seasonal change. If within this zone, the soil may be examined for physical characteristics to determine its suitability under prevalent conditions of climate, traffic, or load for the desired use or for the treatment required to make it suitable. When located below the zone of seasonal change, the soil may be tested to determine its compressibility, elasticity, shear strength, and other properties which control its bearing value.

Zone of Seasonal Change.—The zone of seasonal change is that portion of the earth's crust in which the soils swell when moisture enters and shrink when it escapes, heave owing to frost, lose stability during subsequent thaws, and, in general, are continually changing in properties related to moisture content. The magnitude of these changes is indicated by the studies of F. H. Eno (105). Tests of several hundred soil samples during a 3-year period showed that the moisture content of the upper 24 in. of the campus soil at Ohio State University varies from a minimum of 8.7 to a maximum of 37.8 per cent of the dry weight of the soil. The report of these tests states that "rain, snow, freezing, thawing, wind, sun, and cloud change the top inch or two inches very quickly from about 2 up to 124 per cent of the dry weight of the soil."

The fluctuations in moisture content and related properties are most rapid in the spring during successive periods of freezing and thawing. The spring rains also produce marked changes, although they do not usually affect the soil below a depth of 6 in. Even during dry weather there are some fluctuations, apparently caused by changes in temperature and humidity.

According to the Smithsonian tables (86), the average underground temperature of the earth at a depth of about 3 ft. varies from a minimum of 43°F. in January to a maximum of 90°F. in July, while at a depth of about 20 ft. the variation in temperature throughout the year does not exceed 2°F.

In general, the average soil moisture parallels the average annual rainfall, although there may be local variations. For instance, the moisture content of soils in adjacent fields averages about 10 per cent more than those of soils along the road shoulders, and the soil under pavements is usually wetter than that of the shoulders.

Changes in the direction of moisture movement often affect the stability of soils to a greater extent than variations in moisture



FIG 71.—Device for demonstrating effect of flow of water through sand. (Photograph made when water was flowing upward)

content. For instance, beach sands left in the damp state when the tide has receded may be stable enough to accommodate the fastest automobiles such as use the racing course at Daytona Beach, Fla. Even if such sands are completely immersed, they still may be firm enough to serve as excellent foundations for bridges, buildings, and roads. However, if the moisture flows upward, as from springs or underground water channels, all semblance of stability is lost, and "quicksand" results. Thus, quicksand is in reality a condition and not a material.

However, for equal conditions of upward flow of moisture the greatest loss of stability occurs in sands consisting of very small grains of uniform size.

The effect of the direction of moisture flow on the stability of sands may be demonstrated by the device shown in Fig. 71. This consists of a box so designed that sand can be inundated or subjected to a downward or upward flow of water. In the first two cases the miniature bridge pier is ably supported, but in the third case the upward flow of water changes the stable sand into a quicksand, and the pier rapidly sinks.

A similar loss of stability sometimes occurs in soils of highly capillary properties in freezing climates. It is not known whether this is caused entirely by the liberation of excess moisture during

thaws or whether the upward movement of moisture during the formation of ice crystals is a contributing factor. These soils may become so nearly fluid that the larger particles of gravel or crushed stone in the base course may sink several feet. Burton and Benkelman (103) tested a number of cylinders of soil 1 ft. in diameter and 3 ft. high with a layer of gravel separating the top and bottom halves. After repeated freezings and thawings they found that some of the particles of gravel had sunk as much as a foot. The loss of metal from gravel and macadam road surfaces during spring thaws may thus be explained.

Effect of Structure.—The same soil material, under identical climatic conditions, may undergo a radical change in character when removed from its natural place in the soil profile or when it is broken up, consolidated, or otherwise disturbed. The arrangement of soil particles, which was developed during the ages by varying pressure and other natural constructive and destructive elements, can seldom be reproduced in a short time by artificial means. Disturbance of the natural structure, therefore, often results in changes in those properties of soil which depend on its structure. Thus, in the natural state the soil may be more porous, have higher permeability, and be more stable than the same material in the disturbed state.

Eno (36) found from the tests mentioned that the porosities of soil in the natural state were considerably higher under similar conditions of wetting and drying than those of samples of the same soil in the disturbed state; and it is a well-known fact that with careful tamping the backfilling of a trench will require more soil than that which was excavated. Terzaghi (26) found that the permeability of a sample of clay from Spanish Honduras was 75,000 times greater in the natural than in the disturbed state.

Undisturbed soils are often much more stable than disturbed soils having the same moisture content. Among a number of examples, Casagrande (106) reports the bearing value of a confined clay sample in undisturbed state as being 10 tons per square foot as compared with slightly less than 2 tons per square foot for the same sample at the same density when remolded. The porosity of the undisturbed sample was 54 per cent; that of the disturbed sample capable of sustaining the same load, 45 per cent. In other words, to sustain the same load, greater density was needed in the disturbed than in the undisturbed soil.

The effect of disturbance is not the same in all soils. The structure in some cases is due to certain geological processes and weathering agencies not likely to be repeated. In such cases the soils never regain their former structure once it is destroyed. Other soils, such as the colloidal clays, have developed a type of structure that can be explained only on the basis of certain colloidal surface phenomena. In such cases the disturbed soils may lose their structure but temporarily, there being a gradual return to the former structure, provided the original conditions of pressure, rest, etc., are repeated. A third class, which includes



FIG. 72 — Vertical faces of loess soil in cut.

the cohesionless sands and certain of the silts, seems to have no internal structure and is virtually unaffected by manipulation.

Loess represents those soils having the type of structure that is permanently lost by disturbance.

In the natural state this soil is so resistant to erosion and so stable as to maintain indefinitely almost vertical faces in cuts (see Fig. 72). When removed from the profile and manipulated for use as in a subgrade, it loses stability readily in the presence of moisture and becomes exceedingly troublesome, especially when the frost is leaving the ground. According to Pirsson and Schuchert (42), the stems and roots of successive generations of grasses growing on the loess deposits (formed from dust clouds carried by the wind) and buried by the rising accumulations have by their decay produced the slender vertical tubes responsible for the high vertical cleavage of loess (see Fig. 73).

The performance of putty serves to illustrate the type of structure due to change in colloidal films. When ready for use, this mixture of inert fine soil particles and oil is quite plastic and capable of being deformed by small external pressure. In such state it may be readily forced into crevices and molded on window frames along newly placed glass. After this material has been placed under water for a period of time such as during storage, it changes from the plastic to the semisolid state and crumbles when external force is applied. This indicates that the

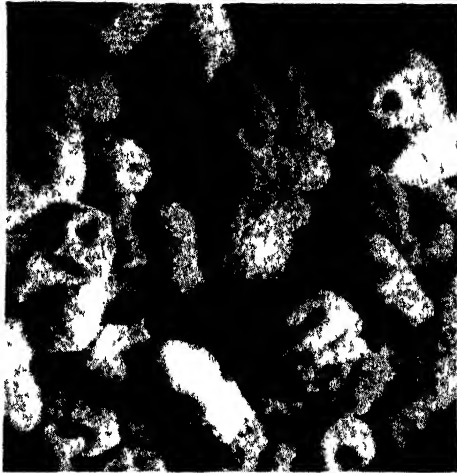


FIG. 73.—Photomicrograph of loess soil particles. $\times 20$.

oil films surrounding the soil particles have undergone a change from the viscous to the semisolid state. That there has been no diminution of the oil binder is indicated by the fact that the plastic state is restored when the putty is removed from the water and manipulated sufficiently between the fingers. There seems to be no limit to the number of times that the material can be changed from the plastic to the semisolid state during storage and from the semisolid to the plastic state by manipulation on removal from storage.

Many soils seem to undergo similar phenomena during alternate periods of rest and manipulation. The moisture contents of many of the natural soils investigated by Professor Eno (35) were far above the plastic limit. Yet the soils in undisturbed state were

more nearly semisolid than plastic. However, it was easy to change such soils into the plastic state by manipulation.

Effect of Moisture Content.—The same soil material in the disturbed state under identical climatic and topographic conditions may perform differently depending upon the moisture content at the time of consolidation and the manner in which the consolidation is accomplished.

During the Bates Road tests (13) subgrade soil more or less dry when covered with the pavement became wetter subsequently than similar soil in a damp or moist condition when the concrete was placed.

In the soils laboratory of the U. S. Bureau of Public Roads, compressed samples of silt and clay soils, which resisted slaking indefinitely if saturated, disintegrated quickly if dry when immersed in water.

Recently R. R. Proctor (38) has shown that earth dams and similar embankments may serve satisfactorily or otherwise, depending on the moisture content and the degree of compaction employed during construction.

Thus, in addition to variations due to difference of materials forming the layers of the profile, the properties of the soil may change radically with seasonal alteration of the layers near the surface caused by climatic changes when removal from the profile changes the natural structure and, finally, when prepared for use by different methods of construction.

Thus soil does not have well-defined and fairly constant properties like other construction materials, such as concrete, steel, brick, and wood. Also, foundation soils react differently from other structures to load. Bridges and buildings fail abruptly when stressed beyond the strength of constituent materials and consequently are designed for safe bearing load. Soils, however, present a different problem. With exceptions, as in the design of earth dams, the prevention of landslides, etc., abrupt failure is not to be anticipated, and resistance to rupture is not an important consideration. Instead, heaving or settlement may damage structures supported by the soil by exceeding certain allowable limits.

Therefore footings are designed for the allowable load, designated by the allowable limit of deflection on the stress-strain diagram, instead of the safe bearing load based on the rupturing

strength of the soil. Those below the line of seasonal change are designed for deflections produced entirely by load. For foundations above this line, settlement and heave due to causes independent of load are also considered.

Deformations Due to Load.—A load placed on the surface of the ground will cause (a) a depression directly under the load and (b) a bulge adjacent to the loaded area. An increase in load causes a corresponding increase in the depression and bulge. The removal of the load will allow the soil to rebound, thus decreasing these deformations. As a soil mass is never a perfectly elastic body, some of the deformation is likely to remain permanently.

Accompanying the surface deformations are a reduction in porosity and a lateral bulging of the undersoil beneath the load. These two changes are not interdependent but are controlled by different sets of laws and may vary widely for different soil types. For some of the firmer soils, sensitive gages capable of measuring to a ten-thousandth of an inch must be used to determine the magnitude of these movements. In the softer soils, however, deformations are often readily discernible. Finally, the same laws do not necessarily control the settlements due to loading both small and large areas.

Therefore in the design of foundations more than simply the stress-strain diagram expressing the relationship of load to deformation is often required. It is also of value to have information on the three structural properties of the soil, which individually or in some combination control the deformations. These properties are compressibility, elasticity, and stability.

Compressibility is the term denoting the tendency of the particles comprising the soil mass to wedge closer together under load, thus decreasing the pore space and causing water or air to be expelled. This type of deformation is more or less permanent (see Fig. 74, middle), and the magnitude and rate of its occurrence are controlled by well-established mathematical laws. Buildings placed on compressible foundation soils can be expected to settle slowly, but such settlements are likely to continue over long periods of time. Abrupt settlements productive of failure are not likely to occur unless excavations for new buildings, construction of drains, etc., provide a new outlet for the moisture in the soil pores. Under such conditions settlements due to compressibility of the undersoil can be great enough to cause the quick

failure of structures which otherwise would probably never have been endangered.

Elasticity is defined as that property of soils which causes them to rebound upon the removal of the load. Elastic soils displace laterally under load and bulge adjacent to the loaded area (see Fig. 74, lower). There is not necessarily a change in the porosity of the soil. Elastic deformations have the character of jellylike distortions which remain only while the soil is loaded.

Stability is defined as the part of the load capacity of the soil which depends upon the shear strength. Shear strength repre-

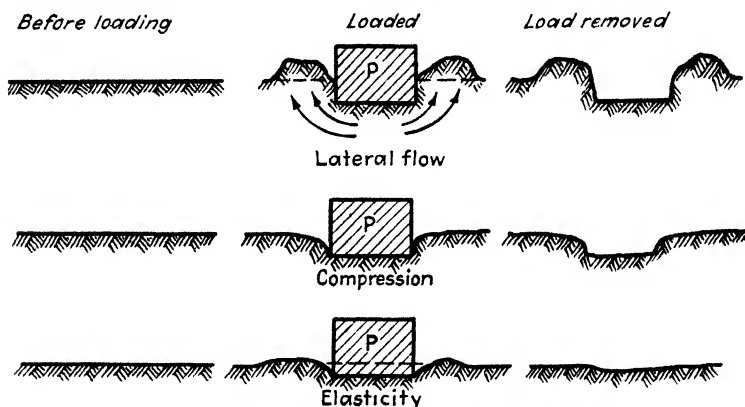


FIG. 74.—Types of settlement of soil under load.

sents the combined effect of internal friction, or the resistance of soil grains to sliding; and cohesion, the stickiness between soil particles. When the shear strength of the soil is exceeded, lateral bulging or flow occurs. In lateral flow (Fig. 74, upper) the soil without change in moisture content squeezes out from beneath the load and causes a bulge in the surface adjacent to the load similar to that of elastic soils. In this case, however, the deformation remains when the load is removed.

Loss of stability accounts for the major failures of earthworks and foundations, including dam failures, landslides, subsidence of fills, overturned bridge piers, and the like (see Fig. 75). Such failures usually occur abruptly. Mathematical relationships for disclosing the characteristics of settlements of soil due to failure in shear have not as yet been developed beyond the controversial stage.



FIG. 75.—Bridge failure caused by settlement of pier.

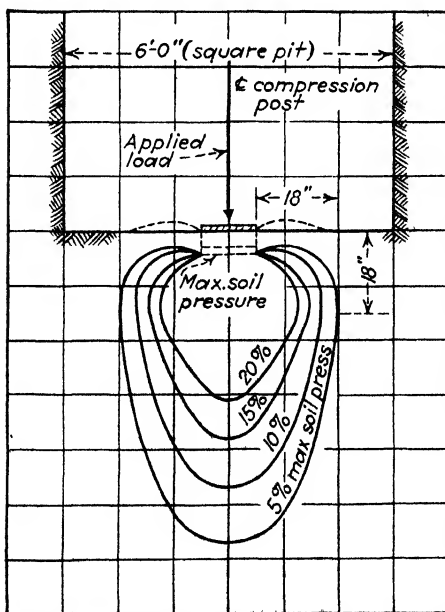


FIG. 76.—Bulbs of pressure in soil under load. (From Housel.)

Bulb of Pressure.—With the formations of depressions in the surface, stresses or pressures productive of deformation occur for some distance below and adjacent to the loaded area. Tests have shown that the pressures under bearing blocks or footings placed on compressible soils have a parabolic distribution. They are greatest at the center and approach zero at the edges. Points of equal pressure intensity take the form of a bulb. Figure 76 shows a series of bulbs of various intensities of pressure beneath a loaded bearing block in an open pit (19). The volume of compressed soil included between the points of zero pressure and the bearing block is known as the "pressure bulb." This is the soil mass which furnishes the support for the block at the surface. The supporting power of the bulb is developed by the resistance to deformation offered by the cohesion and internal friction of the soil.

According to this theory, as developed by Boussinesq (107), when the location at which a load P is applied at the surface is taken as the origin of a system of coordinates, the stress intensity

TABLE 16A.—BOUSSINESQ COEFFICIENTS K CORRESPONDING TO RATIOS OF r/z See Eqs. (38) and (39)

Ratio r/z	Coeffi- cient K	Ratio r/z	Coeffi- cient K	Ratio r/z	Coeffi- cient K	Ratio r/z	Coeffi- cient K
0.00	0.4775	0.90	0.1083	1.80	0.0129	2.70	0.0024
0.10	0.4657	1.00	0.0844	1.90	0.0105	2.80	0.0021
0.20	0.4329	1.10	0.0658	2.00	0.0085	2.91	0.0017
0.30	0.3849	1.20	0.0513	2.10	0.0070	3.08	0.0013
0.40	0.3294	1.30	0.0402	2.20	0.0058	3.31	0.0009
0.50	0.2733	1.40	0.0317	2.30	0.0048	3.50	0.0007
0.60	0.2214	1.50	0.0251	2.40	0.0040	3.75	0.0005
0.70	0.1762	1.60	0.0200	2.50	0.0034	4.13	0.0003
0.80	0.1386	1.70	0.0160	2.60	0.0029	4.91	0.0001

p_z at a point in the ground with vertical and horizontal ordinates of z and r , respectively, is expressed by the relationship

$$p_z = K \frac{P}{z^2} \quad (38)$$

in which

$$K = \frac{3}{2\pi} \frac{1}{[1 + (r/z)^2]^{5/2}} \quad (39)$$

Table 16A supplies values of K corresponding to ratios of r/z for computation purposes. As a numerical example, assume a load of 450 tons applied as a point concentration on the surface. Determine the stress intensity at a point 50 ft. below the load and 15 ft. away horizontally, so that $z = 50$ ft., $r = 15$ ft., and the

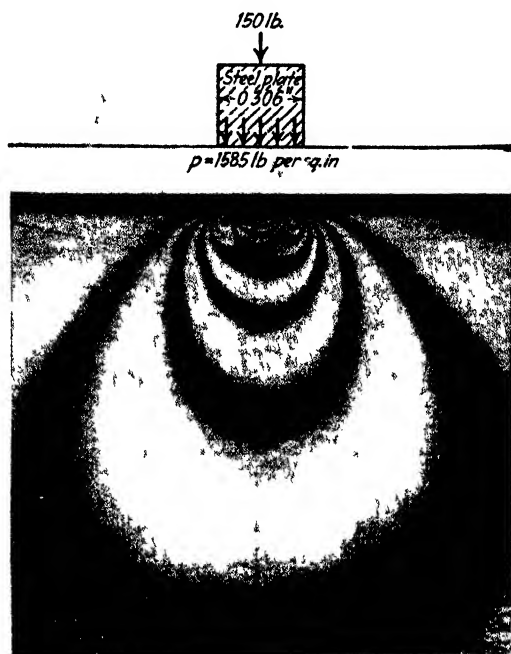


FIG. 77 —Lines of equal vertical pressure in bakelite, simulating bulbs of pressure in soil

ratio $r/z = 0.30$. The corresponding value of K (Table 16A) is 0.385. The stress, in tons per square foot, is then

$$0.385 \times 450 \times 20^2 = 0.0693 \text{ ton per square foot}$$

Photoelastic methods have been used to make visible the "bulb of pressure," or lines of equal vertical pressure in material simulating ground supporting foundations (108). By directing a strong beam of light from large crystals of Iceland spar through a sheet of bakelite (0.308 in. thick) loaded by a rigid block of steel or by a flexible block of bakelite, an image is formed which may readily be recorded on a photographic plate. An example of such an image is shown in Fig. 77 in which the effect of a simple

spread footing is observed. The bands working outward represent diminishing shears or diminishing intensities of stress. They have the same general shapes as the bulbs of pressure in soils.

Observed Settlements.—The Foundation Committee of the American Society of Civil Engineers, in their report on earths and foundations (108), point out that it is erroneous to assume that well-designed foundations undergo no settlement. Among cases of settlement that they cite as conveying more than usual interest

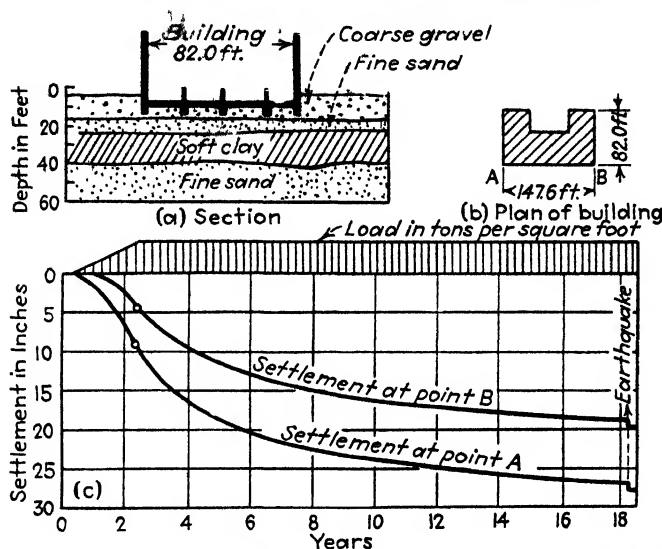


FIG. 80.—Settlement of building due to consolidation of thin layer of soft clay. (Case C.)

are the following, the soil profiles and rates of settlement of which are shown in Figs. 78 to 83 inclusive.

Case A was that of a building 500 by 460 ft. carried on isolated piers resting on piles varying in length from 65 to 75 ft. The surface load varied from 1.5 to 2.5 tons per square foot, and the settlement at the end of 2 years amounted to 20 in. (see Fig. 78).

Case B was that of a group of five oil tanks and a pumping plant placed unsymmetrically in a building and resting on spread footings. The observed settlement of one of the tanks was kept with great accuracy and, although substantial, as shown in Fig. 79, was anticipated, provisions having been made for such movement.

Case C was a post-office building in Austria. A layer of clay about 15 ft. thick lies between two layers of permeable sand, and consolidation caused a settlement of almost 2 ft. (Fig. 80). Piles driven into the sand layer might have proved efficacious in this case.

Case D.—The foundation illustrated in Fig. 81 is one of frequent occurrence in engineering practice. In spite of the deep

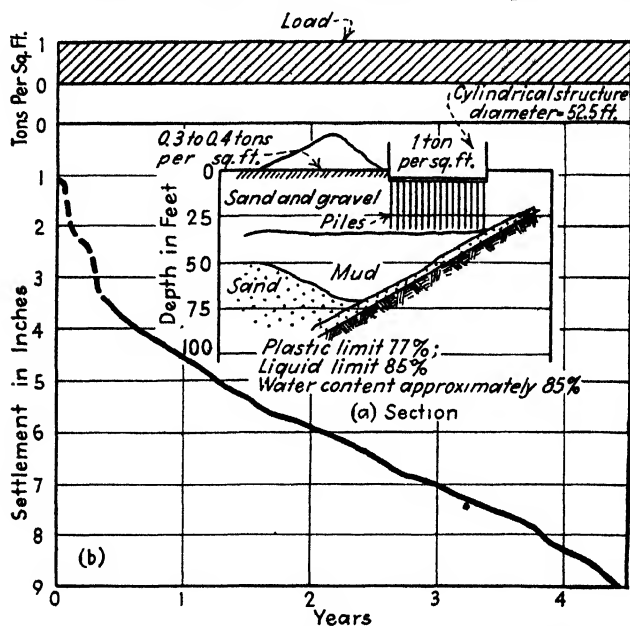


FIG. 81.—Settlement of oil tank on wooden piles due to consolidation of mud layer. (Case D.)

penetration of the piles into sand and gravel, the presence of the mud layer caused a settlement of almost 1 ft. in less than 4 years.

Case E.—The National Theater in Mexico City, Mexico (see Fig. 82), is one of many public buildings that have undergone extraordinary settlements (as much as 5 ft.) due to the slow consolidation of the volcanic ash on which the city is founded. Only the ability to experience a large amount of plastic flow prevented destruction of these buildings due to the dish effect of the settlement.

Case F.—The Washington Monument (Fig. 83) affords the longest settlement record available.

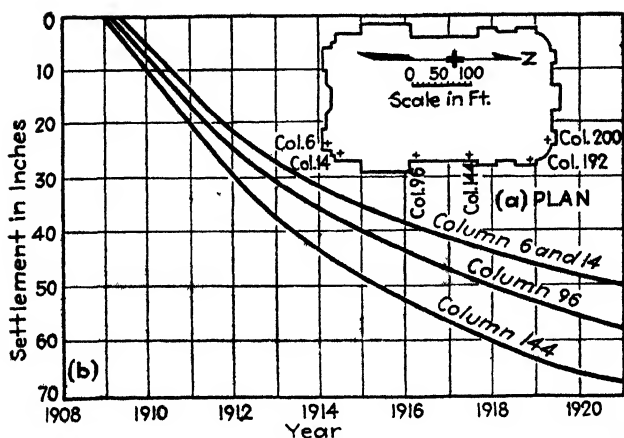


FIG. 82.—Settlement curves of various columns, National Theatre, Mexico City, Mexico. (Case E.)

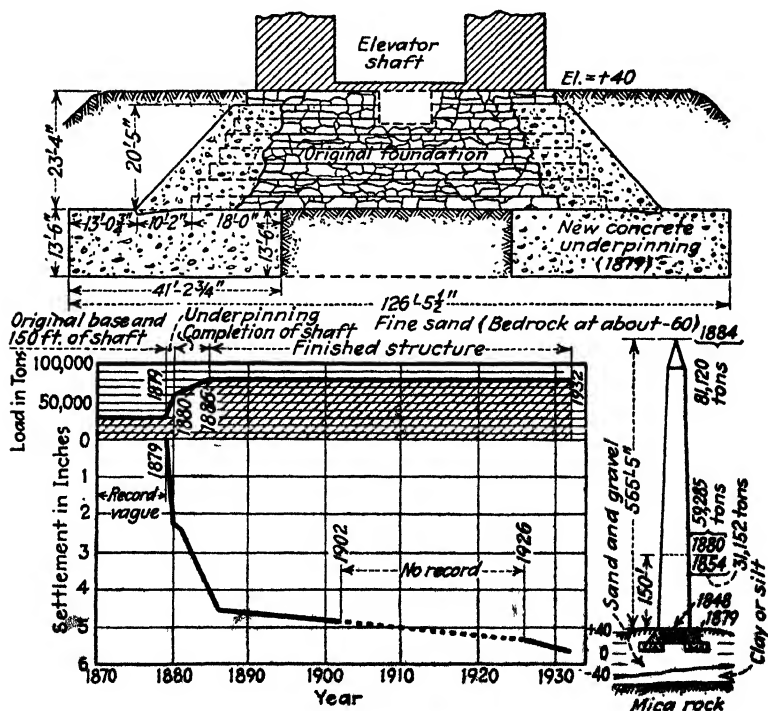


FIG. 83.—Settlement record, Washington Monument. (Case F.)

The original 80-ft.-square foundation was underpinned to an area of 126 ft. $5\frac{1}{2}$ in. square in 1879. Previous to this date, with the structure at a height of 150 ft., the load on the original foundation was about 5 tons per square foot. The monument was built to its full height by 1884, again increasing the load on the enlarged foundation to 5 tons per square foot. No settlement records prior to 1879 are available. Between this date and 1884 the monument settled $4\frac{1}{2}$ in., subsequent to 1885 another inch, and is probably still slowly settling.

It was originally assumed that the underlying stratum was a solid bed of gravel, but borings made in 1931 disclosed a bed of compressible clay, 10 to 40 ft. thick, overlying the rock. The compression of this layer is probably responsible for most of the observed settlement, which fortunately has been quite uniform, with the result that the monument is nearly plumb.

Problems

1. What is meant by zone of seasonal change?
2. Illustrate how the direction of flow of soil moisture may affect the stability of the soil.
3. Discuss the effect of structure on the stability of a soil.
4. Discuss the effect of moisture content on the stability of soils.
5. (a) What controls the allowable load to which foundation soils may be subjected?
(b) How do the principles of design for a foundation resting on soil below the line of seasonal change differ from those for foundations above this line?
6. What are the characteristics of the deformations occurring when a soil is loaded?
7. Define (a) compressibility, (b) elasticity, (c) stability.
8. On what does the shear strength of a soil depend?
9. Explain "pressure bulb."
10. A compressible layer of soil 20 ft. thick is supporting a concentrated load of 3 tons. Using the formula developed by Boussinesq, compute the stress at the surface of the underlying layer at a point directly beneath the load.

CHAPTER IX

SETTLEMENT DUE TO COMPRESSION OF SOIL

During recent years much progress has been made in the development of sound principles and test methods dealing with problems of settlement encountered in the design of bridge footings and other structures erected on soils containing strata of very fine-grained saturated materials. One of the most useful tools in this study has been the laboratory compression test which was developed by Dr. Charles Terzaghi.

Results furnished by this test supplemented with the application of the theory of distribution of pressures in soil indicate within a comparatively short time the compression of saturated strata sandwiched between two more permeable layers or supporting a permeable layer, which would occur only after many years in the field.

Soil Consolidation.—A metal coil spring will compress when a load is placed upon it. Upon removal of the load, the spring will rebound to its original length. Within the elastic limits of the spring, the deflection U is proportional to the load P . Thus,

$$U = \gamma P \quad (40)$$

in which γ is a constant depending upon the dimensions and elastic properties of the metal and the diameter of the coils.

According to formula (40), a spring that is compressed 0.1 ft. by a load P of 3 lb. will be compressed 0.3 ft. by a load of 9 lb.

The term "consolidation," as used in soil mechanics, indicates the complete readjustment of a soil to a given pressure. The amount of consolidation due to the compressibility of the soil depends upon the internal friction of the soil particles, supplemented by the permeability of the soil and the capillary and cohesive properties of the contained moisture films.

Compression of soils is mathematically related to load but logarithmically and not in direct proportion, as in the case of the spring deflection; also, compression of soils occurs more slowly than deflection of springs.

Overburden Produces Settlement.—Soil sediments may undergo a process of consolidation due entirely to the weight of the soil particles within the deposited layer. Deposits of fine-grained soils may be gradually consolidated in this manner from the soft, viscous state at the flocculation limit to approximately the liquid limit. Natural moisture contents of a number of samples of the muck underoil at Four Mile Run on the Mount Vernon Memorial Highway substantiate previous findings by Terzaghi (85) that liquid limits represent approximately the natural moisture contents of old lake or river bottoms and similar sediments (see Table 17).

TABLE 17.—NATURAL MOISTURE CONTENTS AND LIQUID LIMITS OF CORE SAMPLES OF RIVER-BOTTOM MUCK SOILS
In per cent

Core number	Moisture content, natural state	Liquid limit
1	125	132
2	116	112
3	108	103
4	105	96
5	79	62
6	76	77

Compression Device Described.—The essential features of the Terzaghi compression device are illustrated in Fig. 84. A

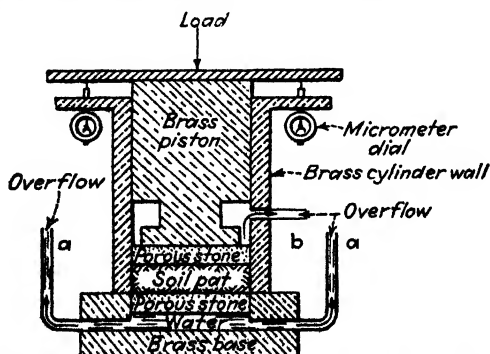


FIG. 84.—Essential elements of Terzaghi compression-test device.

sample of undisturbed soil is placed between two porous stones, the upper one forming the base of a piston which loads the

sample. Water drains freely from both faces of the sample under pressure and passes through the stones, escaping from overflow orifices *a* and *b*. This egress of water reduces the thickness of the sample, which cannot be deformed laterally. The compression of the sample is measured by very sensitive micrometer dials.

Loads are applied in successive increments, each load being about double the preceding one, the maximum being equal to or greater than the proposed pressure to be exerted on the stratum from which the sample was taken. The density of the sample at various stages of the loading is indicated either by the moisture content or by the ratio of volume of pores to volume of solids. The latter is termed the voids ratio of the soil mass. Each load increment is allowed to act until further consolidation of the sample under that load is immeasurable. Thus the voids ratio at any stage of loading indicates the maximum density likely to be produced by the corresponding load.

Samples Carefully Transferred to Testing Device.—In the determination of the load-deformation characteristics of the material in the natural state the samples are transferred to the testing apparatus from the hermetically sealed containers in which the soil was placed at the time of sampling with as little disturbance as possible (70).

A disk of undisturbed material is carefully cut from each core, trimmed to the correct dimensions, and carefully placed in the testing apparatus. This transfer is made in a damp closet to prevent loss of moisture from the sample.

The apparatus used in transferring the sample to the testing device is shown in Fig. 85. It consists of a stand *A* with a slow-motion screw and thrust bearing *B*, a cutter support *C*, a guide ring *D*, a special cutter *E*, a thin metal band *F*, a piano-wire saw *G*, and a plunger *H*.

Cutting the test sample and placing it in the testing device is accomplished by five distinct operations in the following order:

1. A portion of the undisturbed soil is cut from the core as received from the field and placed in the metal band *F*, and the space between the band and the circumference of the soil cake is filled with melted paraffin.

2. The soil sample thus prepared and the cutting apparatus are arranged as shown in Fig. 86.

3. The cutter is forced carefully into the soil cake by means of the slow-motion screw until the soil extends far enough above the top of the cutter to permit it to be smoothed off to the plane of the top of the cutter.

4. The cutter containing the sample is now arranged in the holder as shown in Fig 87, and the portion of soil extending above the cutter is smoothed off carefully to the plane of the top of the cutter by means of the piano-wire saw

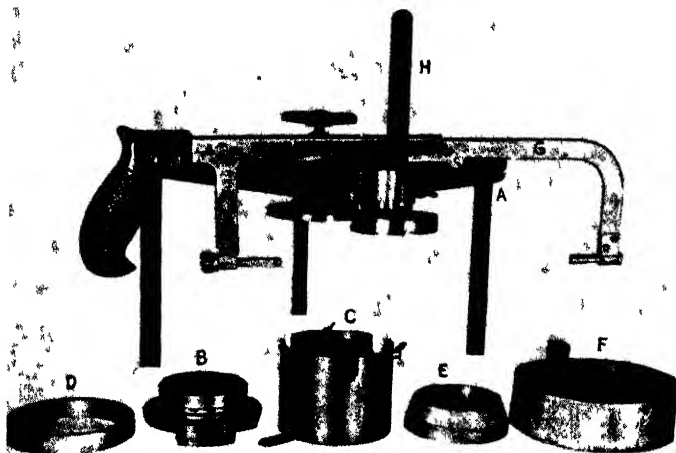


FIG 85 —Essential parts of sample-cutting apparatus.



FIG 86.—Assembly of apparatus for cutting sample from soil core.

5. The sample is then forced slowly into the testing container by means of the plunger *H* (see Fig. 88) and trimmed off flush with the container rim, after which the testing apparatus containing the soil sample is ready to assemble for test as shown in Fig. 84

Test Data Show Load-deformation Relations.—The test furnishes the data shown in Fig. 89. The load-compression curve shows the voids ratio at which the sample will support a given

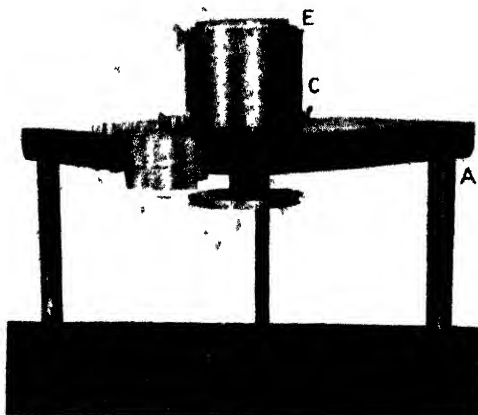


FIG. 87.—Assembly of apparatus for trimming one face of sample flush with cutter rim.

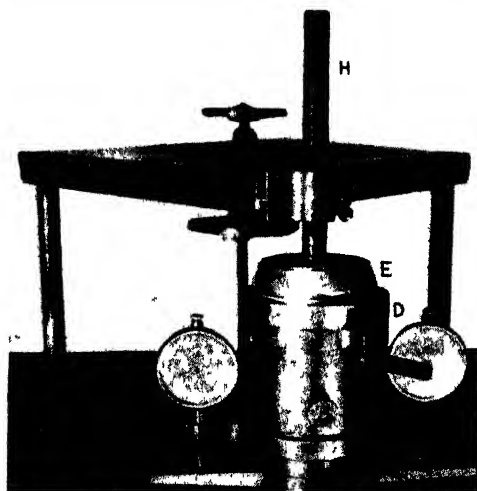


FIG. 88.—Sample being transferred from cutter into compression cylinder.

load. The load-expansion curve shows the relation of increase in voids ratio to decrease of load. The time-compression curve shows the speed at which the test sample consolidates.

According to the load-compression curve, a load of 0.5 ton per square foot compresses the sample to a moisture content of 79.5 per cent ($e = 2.16$). According to the load-expansion curve, decreasing the load from 1.5 tons to 0.5 ton per square foot increases the moisture content from 57.6 ($e = 1.56$) to 61 per cent

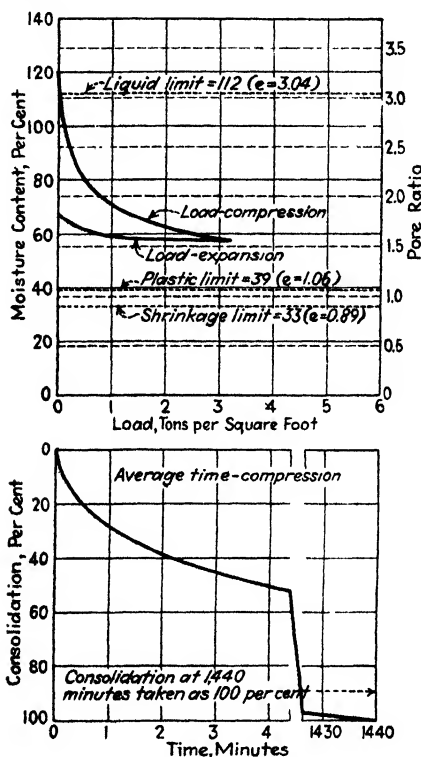


FIG. 89.—Data furnished by compression test of muck soil.

($e = 1.68$). According to the time-compression curve, 28 per cent of the consolidation occurs in 1 min.

For moisture contents above the plastic limit the load-compression relationship is

$$w = \frac{100e}{G} = B - Z \log p \quad (41)$$

in which w = moisture content, per cent of the dry weight.

e = pore ratio.

G = specific gravity of soil solids.

B = moisture content at a load p of 1 ton per square foot.

Z = change in moisture content corresponding to a variation in load represented by one cycle on the logarithmic scale.

For the muck sample (Fig. 89) the load-compression relationship is

$$w' = \frac{100e}{2.71} = 71 - 28.4 \log p$$

It has been previously shown that 100 cu. ft. of the muck soil at the flocculation limit ($w = 269$ per cent, $e = 7.29$) would upon evaporation of moisture become 49 cu. ft. at the liquid limit ($w = 112$, $e = 3.04$) and 25 cu. ft. at the plastic limit ($w = 39$, $e = 1.06$). If all the shrinkage occurred vertically, a deposit of muck soil 100 ft. thick at the flocculation limit would become but 49 ft. thick at the liquid limit and 25 ft. thick at the plastic limit.

The loads required to produce similar consolidations may be obtained by means of the preceding load-compression relationship. That required to compress the deposit to the liquid limit is obtained as follows:

$$\begin{aligned} 112 &= 71 - 28.4 \log p \\ p &= 0.036 \text{ ton per square foot} \end{aligned}$$

The load required to compress the deposit to the plastic limit, as determined in like manner, would be 13.4 tons per square foot.

Load-compression Relationship Discloses Shrinkage Force.—In Chap. V it was shown that the force of capillarity SF (in grams per square centimeter) which causes soils to shrink is related to the pore width according to the expression

$$SF = \frac{0.309}{m} \quad (15)$$

The load 0.036 ton per square foot (36 grams per square centimeter) substituted for SF in this formula gives a value of $m = 0.0086$ cm. This can be considered as the effective pore width of the muck soil at the liquid limit. In a similar manner the effective width of the pores of the muck soil at the plastic limit is found to equal 0.000023 cm.

When Compression Tests Are of Value.—From information furnished by borings made at the foundation site it can be learned (a) if rock or other solid material is present at a depth to which it would be practical to extend the foundation; (b) if strata consisting of granular materials such as well-compacted sands and gravels, preferably extending to rock, are located close enough to the surface so that the foundations can be placed directly upon them; (c) if the conditions for penetration and lateral support are such that a pile foundation can be extended to rock, compacted sand or gravel, or other incompressible soil; or (d) if the profile is of such character that the foundation must be carried, at least in part, by a layer of compressible soil which in time will result in settlement of the supported structure. It is only in the last case that tests to disclose the compressible properties of the soil are required.

Deformation Coefficients Defined (109–110).—When a load is applied to a prism of saturated clay sandwiched between two filters, as in the compression test, or between two pervious layers in the soil profile, at the first instant the water takes all the load. As the water moves vertically and escapes through the filters in the case of the testing device, or through the pervious layers in the soil profile, the load is gradually transferred more and more to the skeleton formed by the soil solids. This continues until the soil skeleton supports the entire load, when equilibrium is reached, and the escape of water ceases.

The total settlement Q due to consolidation of a soil layer under load is

$$Q = D_1 \frac{e_1 - e_2}{1 + e_1} \quad (42)$$

in which D_1 = thickness of soil layer under pressure p_1 .

e_1 = voids ratio of soil at pressure of p_1 .

e_2 = voids ratio of soil at equilibrium after pressure has been increased from p_1 to p_2

This formula holds when either one or both of the layers bounding the compressible layer are pervious.

Several other constants are used in mathematical explanations of the mechanics of consolidation. They are:

1. The coefficient of compressibility a , which is defined as the average decrease in voids ratio for each gram per square centi-

meter increase in pressure. It is obtained by the expression

$$a = \frac{e_1 - e_2}{p_2 - p_1} \quad (43)$$

If $e_1 = 2.50$ and $e_2 = 1.36$, the corresponding pressures for the muck soil will be $p_1 = 0.1766$ ton per square foot (177 grams per square centimeter); $p_2 = 5.336$ tons per square foot (5,336 grams per square centimeter). The corresponding coefficient of compressibility then becomes

$$a = \frac{2.50 - 1.36}{5,336 - 177} = 0.000221$$

2. The average coefficient of permeability k , which is defined as the average velocity of water during a change in voids ratio from e_1 to e_2 under a hydraulic gradient of 1, *i.e.*, when the loss in head between two points in a soil column equals the distance d between the points.

3. The reduced coefficient of permeability k_0 (in centimeters per minute), which is the velocity that the water would have if the loss of head were d_0 instead of d .

4. The coefficient of consolidation c , which is defined as the ratio of k_0 to the coefficient of compressibility a . It is obtained by the expression

$$c = \frac{k_0}{a} = \frac{2k}{a(2 + e_1 + e_2)} = \frac{2k(p_2 - p_1)}{(e_1 - e_2)(2 + e_1 + e_2)} \quad (44)$$

in which k = average coefficient of permeability during a change in pore ratio from e_1 to e_2 .

Then

$$k_0 = \frac{2k}{2 + e_1 + e_2} \quad (45)$$

According to the laboratory determination, the average coefficient of permeability of the muck soil during compression from $e_1 = 2.50$ to $e_2 = 1.36$ equals 9.42×10^{-6} cm. per minute. Then

$$c = \frac{2 \times 9.42 \times 10^{-6}(5,336 - 177)}{(2.50 - 1.36)(2 + 2.50 + 1.36)} = 0.01455$$

Stress Distribution Varies with Kind of Loading.—There may be three different types of pressure distribution within a layer of undersoil: uniform, triangular, and trapezoidal. Thus, in the soil

profile (Fig. 90) let it be assumed that either the thickness of the compressible layer is so small or the effect of a superimposed uniform load is so much greater than that of the weight of the soil within the layer that for practical purposes the weight of the soil

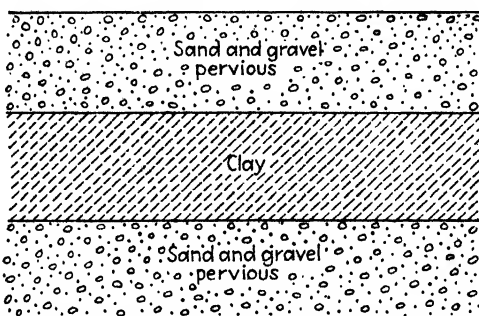


FIG. 90.—Soil profile comparable to test conditions.

within the layer can be disregarded in computing the pressures. Then we have the condition of uniform pressure top to bottom, which is represented by the rectangle (Fig. 91A).

If, on the other hand, there is no superimposed load or soil layer above the compressible layer, then all the pressures within the layer are due to weight of the soil alone and in consequence

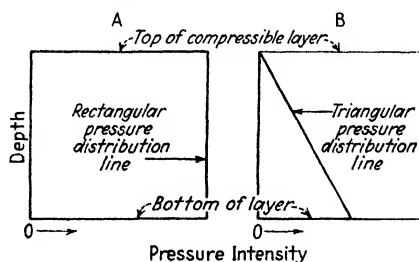


FIG. 91.—Rectangular and triangular pressure distributions.

would be triangular (Fig. 91B). The triangular distribution may also be such that the apex of the triangle is at the bottom of the compressible layer, instead of at the top as shown in Fig. 91B. Limited on one side by the rectangular loading and on the other by the triangular loading, there may be any condition of trapezoidal loading. The times at which any percentage of total consolidation occurs, as determined from the time-compression curve in the manner just discussed, applies only for the uniform

condition of loading. Under any other condition the times of settlement are obtained by qualifying the time required for uniform condition of loading by a constant.

Theoretical Time-compression Relationship Varies with Stress Distribution.—The time-compression curve (Fig. 89) shows the effect of both the permeability of the soil and the internal adjustment of the particles. The curve of relation of time to that part of the compression due to permeability alone generally coincides with the time-compression curve for about 85 to 90 per cent of the total theoretical compression.

The fraction of the compression due to permeability is designated by the letter q .

Theoretically, the reduction in pore ratio q in percentage of the total reduction in pore ratio ($e_1 - e_2$) at any elapsed time t after the load has been increased from p_1 to p_2 , the pore ratio e of the sample at the time t , the thickness d_0 at a pore ratio of $e = 0$, and the coefficient of consolidation c are related logarithmically. For uniform pressure distribution (Fig. 91A) the expression is

$$q = \frac{(e_1 - e)}{e_1 - e_2} \\ = 1 - \frac{8}{\pi^2} \left[e^{-J} + \frac{1}{9} e^{-9J} + \frac{1}{25} e^{-25J} + \dots + \frac{e^{-(2n+1)^2 J}}{(2n+1)^2} \right] \quad (46)$$

Formula (46) expresses the relation of percentage of consolidation q to a period of duration of load, t , for any condition of pressure distribution—rectangular, triangular, and trapezoidal—when the soil being compressed is sandwiched between two filters or two permeable soil layers. When water is free to flow from only one face of the soil being compressed, formula (46) expresses the relation of q to t only when the pressure distribution is rectangular or uniform. For a test sample

$$J = \frac{\pi^2 ct}{d_0^2} = \frac{\pi^2 t}{d_0^2} \times \frac{2(p_2 - p_1)k}{(e_1 - e_2)(2 + e_1 + e_2)} \quad (47)$$

When a definite numerical value is assigned to q it becomes q_1 . Then J has a definite value designated as J_1 as shown later in Table 18.

Thickness of Strata Influences Speed of Compression.—From the expression for J it can be seen that the time t required for

equal percentages of consolidation q varies as the square of the thicknesses of the soil squeezed.

The relative periods of load application producing equal percentages in soil strata and their representative laboratory samples, therefore may be expressed by equations as follows:

Condition A.—Water free to flow from both top and bottom of the soil stratum:

$$t_D = \frac{t_d \times D_1^2}{d_1^2} \quad (48)$$

where d_1 = thickness of soil sample at a voids ratio of e_1 .

$$= (1 + e_1)d_0 \quad (49)$$

d_0 = thickness of soil sample at a voids ratio of 0.

D_1 = thickness of soil layer at a voids ratio of e_1 .

t_d = time required for sample to consolidate from e_1 to e_2 .

t_D = time required for soil layer to consolidate from a voids ratio of e_1 to e_2 .

Condition B.—Water free to flow out of but one face, either top or bottom, of the soil stratum:

$$t_D = \frac{t_d \times 4D_1^2}{d_1^2} \quad (50)$$

Thus the effect of a filter on but one side of a compressible layer has the same effect on the rate of consolidation as two filters or pervious layers have on a layer twice as thick. Subsequent examples are based on the findings of the Special Committee on Foundations of the A.S.C.E. (110) and are limited to the condition of a layer bounded by permeable sand on top and impervious rock below the compressible layer.

Pressure Distribution.—There may be five general cases of pressure distribution for condition *B* of but one permeable boundary, as follows:

Case 1.—Uniform, or rectangular.

Case 2.—Triangular, with zero pressure at the top of the compressible layer.

Case 3.—Trapezoidal, with the greater pressure at the bottom of the compressible layer.

Case 4.—Triangular, with zero pressure at the bottom of the compressible layer.

Case 5.—Trapezoidal, with the greater pressure at the top of the compressible layer.

Gilboy (110) has determined values of J and q for cases 2 and 3 and Kimball (111) for cases 4 and 5. Complete investigation of all five cases, including derivations of all steps used and the mathematical development, has been made by Palmer and Barber (112).

The important cases of trapezoidal distribution of pressure, which are often met in actual practice, are solved by means of interpolating between the two extreme cases of uniform and triangular pressure distribution for values of J .

This makes possible a simplified method of utilizing the actual time-compression curve for computing rates of settlement under pressure distributions coming under the foregoing cases within the range of agreement between the determined and theoretical curves.

If t is the time for a certain percentage consolidation q under uniform load, t_a the time for the same q under any other loading, J_1 the factor for uniform distribution, and J_2, J_3 , etc., the factors for the other loadings, then

$$J_1 = \frac{\pi^2 ct}{4d_0^2}, \quad J_2 = \frac{\pi^2 ct_a}{4d_0^2}, \quad \text{and} \quad J_3 = \frac{\pi^2 ct_a}{4d_0^2}$$

and

$$\frac{t_a}{t} = \frac{J_2}{J_1} = \frac{J_3}{J_1} \quad (51)$$

or

$$t_a = C_1 t = C_2 t$$

when

$$\frac{J_2}{J_1} = C_1 \quad \text{and} \quad \frac{J_3}{J_1} = C_2 \quad (52)$$

Values of J_1 and J_2 are given in Table 18 and values of C_1 and C_2 in Table 19.

The interpolation factor for trapezoidal stress distribution depends upon the ratio u of the stress at the top to the stress at the bottom of the layer. For any assignable value of q , the time t for uniform stress distribution is determined from the time-compression curve and either formula (48) or (50), depending on which is applicable. The time required for the same q , when the stress distribution is trapezoidal, is then easily determined from formula (52), the constant C being obtained from an enlarged form of Table 19.

TABLE 18.—VALUES OF J_1 FOR UNIFORM DISTRIBUTION AND J_2 FOR TRIANGULAR DISTRIBUTION FOR EQUAL PERCENTAGES OF CONSOLIDATION q

Consolidation q	Numerical values of	
	J_1	J_2
0	0.00	0.00
10	0.02	0.12
20	0.08	0.25
30	0.17	0.39
40	0.31	0.55
50	0.49	0.73
60	0.71	0.95
70	1.00	1.24
80	1.40	1.64
90	2.09	2.35
100	∞	∞

TABLE 19.—VALUES OF CONSTANT USED IN DETERMINING RATES OF SETTLEMENT OF TRAPEZOIDAL PRESSURE-DISTRIBUTION SYSTEMS, WHEN LOWER FACE OF COMPRESSIBLE LAYER IS IMPERVIOUS

Percentage of consolidation q	C_1 for values of $u < 1$:					
	0.9	0.8	0.6	0.4	0.2	0
5	1.6	2.32	4.0	6.0	8.6	12.0
20	1.12	1.25	1.6	2.0	2.5	3.1
40	1.04	1.10	1.2	1.4	1.6	1.8
60	1.02	1.04	1.1	1.2	1.2	1.3
80	1.01	1.02	1.0	1.1	1.1	1.2
95	1.01	1.02	1.0	1.0	1.1	1.1

Percentage of consolidation q	C_2 for values of $u > 1$:					
	∞	10.0	5.0	3.0	2.0	1.5
5	0.26	0.43	0.54	0.66	0.78	0.87
20	0.27	0.44	0.55	0.67	0.79	0.88
40	0.39	0.53	0.63	0.73	0.82	0.90
60	0.59	0.69	0.75	0.82	0.88	0.93
80	0.77	0.82	0.86	0.90	0.93	0.96
95	0.91	0.93	0.94	0.96	0.97	0.98

Size and Shape of Loaded Area Important.—The manner in which the distribution of stress differs for different conditions of loading can be illustrated by several examples. Consider a soil profile consisting of 10 ft. of noncompressible sand, which rests on a clay layer 5 ft. thick, which is in turn underlain by impervious rock. The clay layer is assumed to be completely consolidated under its own weight but only 50 per cent consolidated under the weight of the sand layer at the time of external loading. All soil weights are considered as 100 lb. per cubic foot.

If the clay layer is assumed to have attained complete consolidation due to existing pressure, then the top of the clay is under an inactive pressure of 1,000 lb. per square foot, or the weight of 10 ft. of sand. The bottom of the clay is under an inactive pressure of 1,500 lb. per square foot on the same basis. However, on the assumption that but 50 per cent of the consolidation due to the sand has been completed, a portion of that pressure becomes active and causes increased settlements. Details of handling such a problem are ably treated elsewhere (110). In this particular problem the pressure of the sand is divided into two equal parts, as shown by the stress diagrams (Fig. 92), the division being made by the diagonal drawn from the upper right- to the lower left-hand corner. The pressures to the left of this line are inactive, and those to the right are active, at the time of external loading. Consequently the clay layer has been consolidated by a pressure that is 1,000 lb. per square foot at the top and 500 lb. per square foot at the bottom of the layer.

Example 1. Uniform Loading over Infinite Area.—Consider the preceding soil profile subjected to a uniform load of 6,000 lb. per square foot. This load on top of the sand layer creates an active stress of 6,000 lb. per square foot throughout the entire depth of the clay layer, since no bulb of pressure could exist under an infinite loaded area. Consequently the total stress at the top of the clay layer is $6,000 + 1,000 = 7,000$ lb. per square foot, of which 6,000 is active and 1,000 inactive. The total stress at the bottom of the clay layer is $6,000 + 1,000 + 500$ or 7,500 lb. per square foot, of which only 500 is inactive and 7,000 active. This stress diagram is shown in Fig. 92A.

Example 2. Circular Loading.—Consider a load of 6,000 lb. per square foot, confined to a circular area 10 ft. in diameter.

The maximum stress in any horizontal plane beneath the load will occur vertically under the center of the area. It is computed from the formula

$$p_z = p_0(1 - \cos^3 \alpha) \quad (53)$$

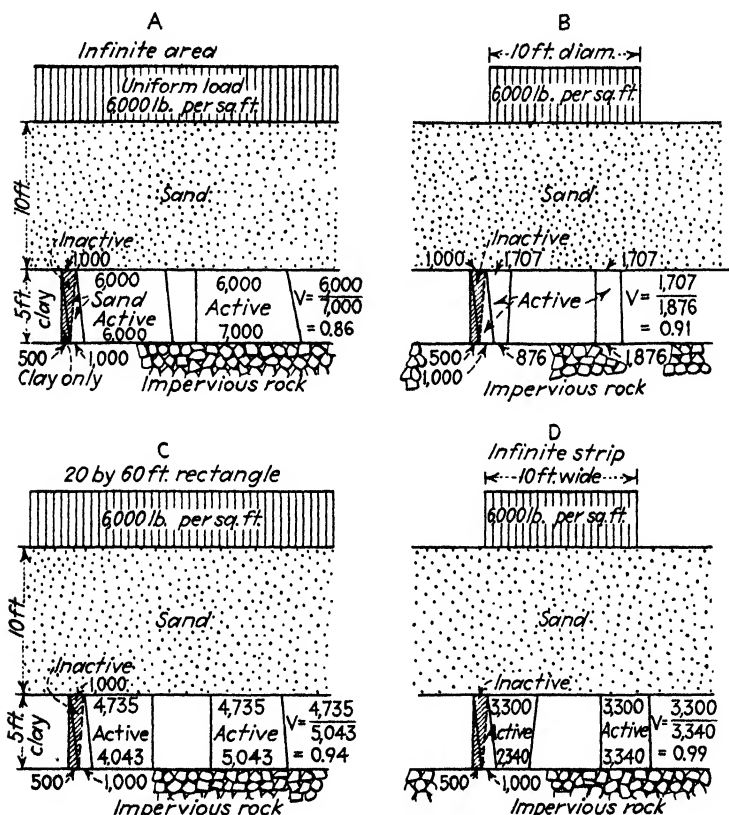


FIG. 92.—Maximum pressure distributions under four types of loading areas.

where p_z = stress intensity at any point vertically under the center of the area.

p_0 = unit load, 6,000 lb. per square foot in this case.

α = angle formed by vertical axis of loaded area and line drawn from point on this axis where stress intensity is being computed to any point on circumference of circular area.

When the point is at the top of the clay layer, α is $26^{\circ}34'$; when it is at the bottom of the clay layer, $\alpha = 18^{\circ}26'$. Substituting in formula (53), p_z at the top of the clay layer is 1,707 lb. per square foot and at the bottom of the clay layer is 876 lb. per square foot. The total pressure at the top of the clay layer becomes 2,707 lb. per square foot, of which 1,707 is active, and at the bottom of the clay it is 2,376 lb. per square foot, of which 1,876 is active. This stress distribution can be represented as a straight line, as shown in Fig. 92B.

Example 3. Rectangular Loading.—Consider that the load applied to the preceding soil profile is 6,000 lb. per square foot,

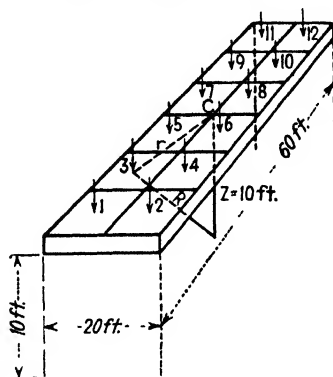


FIG. 93.—Subdivision of rectangular loading area for determining stress distribution.

limited to a rectangular footing 60 ft. long and 20 ft. wide. Again, the maximum stress intensity in any horizontal plane will occur vertically under the center of the loaded area. The method of subdivision is usually employed in determining this stress.

Divide the footing into 12 equal squares as shown in Fig. 93. Each square has an area of 100 sq. ft., and the total load in each is 600,000 lb. This is considered as a point load acting at the middle of each square. From formulas (38) and (39) (Chap. VIII) the intensity of the stress produced by each point load at the point in the top and bottom of the clay layer vertically under the mid-point of the footing can be determined. In the present example three computations of the constant K [formula (39)] suffice, since the whole system is symmetrical in both directions. These are summed up in Table 20. As each computation holds good for four squares, combined stress intensity at a point on the top of

the clay layer 10 ft. below the center of the footing is

$$p = 0.1973 \times 4 \times \frac{600,000}{100} = 4,735 \text{ lb. per square foot} \quad (54)$$

Similarly, the stress at a point on the bottom of the clay 15 ft. below the center of the footing is computed to be 4,043 lb. per square foot. Thus the total maximum pressure at the top of the layer is 5,735 lb. per square foot, of which 4,735 is active, and the total maximum pressure at the bottom of the layer

TABLE 20.—COMPUTATIONS OF VERTICAL STRESSES BELOW A 10- BY 30-FT. FOOTING

Load numbers (see diagram)	Distance r , feet	Ratio r/z	Coefficient K
5, 6, 7, 8.....	$5\sqrt{2}$	$\sqrt{2}/2 = .707$	0.1733
3, 4, 9, 10.....	$5\sqrt{10}$	$\sqrt{10}/2 = 1.58$	0.0209
1, 2, 11, 12.....	$5\sqrt{26}$	$\sqrt{26}/2 = 2.55$	0.0031
Total	0.1973

is 5,543 lb. per square foot, of which 5,043 is active. This stress distribution is shown in Fig. 92C.

Example 4. Strip Loading.—The maximum stress under a long, narrow loaded strip is directly below the center line of such a strip. Loading the profile under discussion with a uniform load, 6,000 lb. per square foot confined to a long, narrow strip 10 ft. wide produces a stress distribution somewhat similar to that of the rectangular loaded area. The stress intensity is computed from the formula

$$p_z = Kp_0 \quad (55)$$

in which p_z = stress intensity at any point z ft. below the center of the strip.

p_0 = unit load at the surface.

K = a constant determined from the formula

$$K = \frac{1}{\pi} \left(\sin^{-1} \frac{2u}{1+u^2} + \frac{2u}{1+u^2} \right) \quad (56)^1$$

¹ The values of K have been worked out for various values of u by the Special Committee on Foundations of the A.S.C.E. (110).

in which

$$u = \frac{b}{z},$$

where b = one-half width of strip.

z = depth below load.

In the example under discussion K is 0.55 when $z = 10$ ft. and 0.39 when $z = 15$ ft. Thus the pressure intensities produced by the strip loading are 3,300 lb. per square foot at the top and 2,340 lb. per square foot at the bottom of the layer. Thus the total pressure at the top of the clay layer is 4,300 lb. per square foot, of which 3,300 lb. per square foot is active, and the total pressure at the bottom of the clay layer is 3,840 lb. per square foot, of which 3,340 is active. Figure 92D shows this stress distribution.

Total Settlement.—The total settlement of a compressible layer under load can be computed from formula (42). The load-compression curve (Fig. 89) shows the relation between load and voids ratio as determined by test. On the assumption that this is representative of the clay layer being used as an example in this discussion, the determination of the voids ratio becomes a simple matter, once the pressures are computed.

In all the four examples of loading, the clay layer prior to loading has been consolidated under the stress distribution labeled "inactive" in Fig. 92. According to this, the top of the clay layer should be at the voids ratio that corresponds to a load of 1,000 lb. per square foot on the load-compression curve, and the bottom of the layer should be at the voids ratio corresponding to 500 lb. per square foot prior to loading. From the load-compression curve, it is seen that the top of the clay layer is at voids ratio 2.23, and the bottom at voids ratio 2.51. The average of these is 2.37, which may be taken as the value e_1 for all examples when determining total settlements from formula (42).

Example 1. Total Settlement under Uniform Loading.—The stress distribution for this example showed that with the addition of a uniform load of 6,000 lb. per square foot the total pressure in the top of the clay layer was 7,000 lb. per square foot. Thus the top of the layer should ultimately attain a voids ratio corresponding to a load of 7,000 lb. per square foot on the load-compression curve. This value, according to Fig. 89, is 1.53. In the same manner the voids ratio of the bottom of the layer will eventually

be 1.51, or that produced by a load of 7,500 lb. per square foot acting for a long time. The average is now 1.52, which is the value e_2 for this example. Since the layer is 5 ft. thick prior to loading, the total settlement will be

$$Q = \frac{2.37 - 1.52}{1 + 2.37} \times 5 = 1.26 \text{ ft., or } 15.12 \text{ in.}$$

Example 2. Total Settlement under Circular Loading.—According to the previous computations of stress for this example, the top of the clay was under a total pressure of 2,707 lb. per square foot, and the bottom under a total pressure of 2,376 lb. per square foot. The voids ratio at the top is found ultimately to reach 1.79, and at the bottom to reach 1.85. The average voids ratio e_2 of the layer is thus 1.82. The total settlement is

$$Q = \frac{2.37 - 1.82}{1 + 2.37} \times 5 = 0.82 \text{ ft., or } 9.84 \text{ in.}$$

Example 3. Total Settlement under Rectangular Loaded Area. The total pressures in this case were previously determined to be 5,735 lb. per square foot at the top and 5,543 lb. per square foot at the bottom of the clay layer. These pressures are capable of compressing both the top and bottom of the clay layer to a voids ratio of 1.57 as read from the load-compression curve (Fig. 89). This average voids ratio indicates a total settlement of

$$Q = \frac{2.37 - 1.57}{1 + 2.37} \times 5 = 1.19 \text{ ft.} = 14.28 \text{ in.}$$

Example 4. Total Settlement under Strip Loading.—This particular example showed total pressures of 4,300 lb. per square foot at the top and 3,840 lb. per square foot at the bottom of the clay layer. These are capable of ultimately compressing the top and bottom of the layer to voids ratios of 1.64 and 1.66, respectively, an average of 1.65. This is indicative of a total settlement of

$$Q = \frac{2.37 - 1.65}{1 + 2.37} \times 5 = 1.07 \text{ ft., or } 12.84 \text{ in.}$$

Rate of Settlement.—The rate of settlement of loaded layers is dependent on three factors: thickness of layer, number of faces free to drain, and the distribution of the active pressures. The relations of these have been expressed previously. When dealing

with uniform stress distribution one needs only the curves of Fig. 89 to supplement the pressures in computing rates of settlements. Trapezoidal pressure distributions require a correction coefficient to the time determined for the same degree of consolidation for the uniform pressure.

The four examples given in this discussion result in trapezoidal pressure distributions. The correction coefficients are determined as given in the discussion of Table 19.

Suppose that the layer in these examples had been loaded so that the combined active pressures were uniform and equal at all depths in the layer. Then the time necessary to complete 20 per cent of the consolidation could be computed directly from formula (50) and the time-compression curve (Fig. 89), which gives t_d as 0.44 min. for that consolidation in the laboratory. The thickness d_1 is 0.34 in., corresponding to a voids ratio of 2.37. Thus

$$t_D = \frac{0.44 \times 4 \times (5 \times 12)^2}{0.34^2} = 54,600 \text{ min., or } 0.104 \text{ yr.}$$

In example 1, the active pressure varied from 6,000 lb. per square foot at the top of the clay layer to 7,000 lb. per square foot at the bottom of the clay layer. The ratio u is thus $6,000/7,000 = 0.86$. From Table 19, the value of the constant C is interpolated as 1.17, which means that for that particular stress distribution it takes 1.17 times as long to attain 20 per cent consolidation as was necessary in the theoretical uniform stress distribution. Consequently, 20 per cent of the total settlement occurring in example 1 will take place in $0.104 \times 1.17 = 0.12$ year.

In a similar manner the times of 20, 40, 60, and 80 per cent consolidations for all four examples were determined as given in Table 21.

Natural Structure Contributes to Bearing Value.—The effect of natural structure of a soil on its compression is illustrated by the Figs. 94 and 95. The full-line curves represent the load-compression relations of the muck soil in undisturbed state. The broken lines represent the same relations for the soil in disturbed state. The equations expressing these relations are

$$e = 2.11 - 1.47 \log p \text{ (undisturbed state)}$$

$$e = 1.46 - 0.99 \log p \text{ (disturbed state)}$$

The compressed state of the undisturbed sample at the beginning of the test is disclosed by the horizontal portion of the full-line

TABLE 21.—COMPUTED RATES OF SETTLEMENT

Per cent consoli- dation	Theo- retical, $C = 1$	Example 1, uniform load $u = 0.86$			Example 2, circular load $u = 0.91$			Example 3, rectangular load $u = 0.94$			Example 4, strip load $u = 0.99$		
		Settle- ment, inches	C_1	Time, years	Settle- ment, inches	C_1	Time, years	Settle- ment, inches	C_1	Time, years	Settle- ment, inches	C_1	Time, years
20	0.104	3.02	1.17	0.12	1.97	1.1	0.11	2.86	1.07	0.11	2.57	1.01	0.10
40	0.54	6.04	1.06	0.57	3.94	1.04	0.56	5.71	1.02	0.55	5.13	1.00	0.54
60	1.39	9.07	1.03	1.43	5.90	1.02	1.42	8.57	1.01	1.40	7.70	1.00	1.39
80	3.91	12.10	1.01	3.95	7.87	1.01	3.95	11.42	1.01	3.95	10.27	1.00	3.91
100	15.12	9.84	14.28	12.84

curve (Fig. 94) at a voids ratio $e = 3.3$. According to the first equation, the load p at this voids ratio for the undisturbed sample equals 0.156 ton per square foot. The corresponding load for the disturbed sample would be 0.0138 ton per square foot—less than one-tenth as much. The liquid limit of this soil is 112 per cent ($c = 3.0\%$), and the plastic limit is 51 per cent ($e = 1.38$). According to the curves in Fig. 95, the soil at the plastic limit would support a load of 3.15 tons per square foot if undisturbed and a load of 1.25 tons per square foot if disturbed. The effect of the difference in compression on the settlement of a layer is indicated as follows:

Let it be assumed that layers of both disturbed and undisturbed soil 10 ft. thick at the liquid limit are loaded at the rate of 1 ton per square foot. The disturbed layer will be

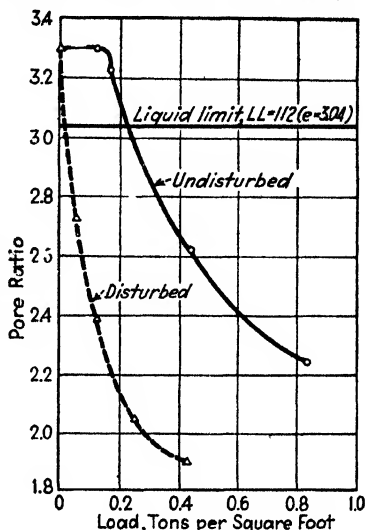


FIG. 94.—Relation of pore ratio to load for a soil in undisturbed and disturbed states.

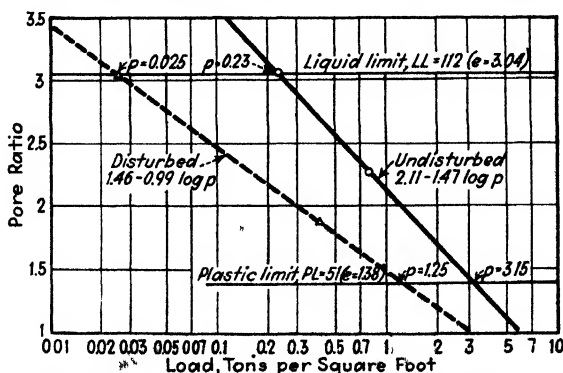


FIG. 95.—Load-compression relations for soil in undisturbed and disturbed states.

decreased to a thickness of $\frac{1 + 1.46}{1 + 3.04} \times 10 = 6.1$ ft., causing a

settlement of 3.9 ft. The undisturbed layer will be reduced to a thickness of $\frac{1 + 2.11}{1 + 3.04} \times 10 = 7.7$ with a corresponding settlement of 2.3 ft. As the load increases, the two curves (Fig. 95) draw closer together owing to the fact that under increasing consolidation the effect of structure diminishes. Eventually all effect of structure will be overcome so that all of the support will depend upon the mechanical characteristics of the grains alone. Then both disturbed and undisturbed soil will support the same

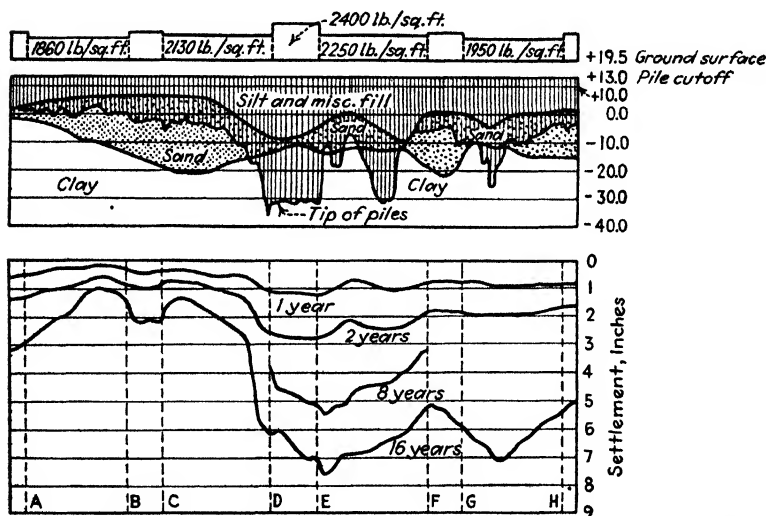


FIG. 96.—Effect of driving piles on settlement of building foundation. (From Casagrande.)

load. The load at which this occurs in this case is theoretical and equals 22.5 tons per square foot. The corresponding moisture content is imaginary and equals 4.8 per cent ($e = 0.13$).

The effect of soil structure on the design of foundations is a controversial question. It has been suggested that piles driven into undisturbed clay layers tend to reduce the supporting value of the soil, making it less stable than if raft footings were used. Casagrande (106) describes an interesting case of a group of buildings founded on Boston clay, which discloses that the driving of piles into undisturbed clay increases the settlements of structures resting on it. Figure 96 shows the distribution of

the loads, the soil profile, the depth to which piles were driven, and the settlements at different periods after construction. The layer of clay was fairly homogeneous, approximately of uniform compressibility, except for a very soft layer above elevation -10 at the extreme left, and from 55 to 85 ft. thick. He analyses the settlements as follows:

Taking into consideration the differences in the thickness of the clay layer and the differences in load, the settlement curve after 16 years should show less variation, and the maximum settlement should be smaller than is represented by the actually observed settlement curve. Owing to the disturbance of the clay structure, the settlement is much larger in those sections where the piles penetrate into the clay. The effect of the disturbance spreads a short distance beyond the region of the disturbance, so that the undisturbed regions between zones of disturbance show also larger settlements. This is partially due to a decrease in lateral support of the undisturbed clay mass.

The considerable settlement on the extreme left is due to the less consolidated clay above elevation -10 and a slight load concentration. The depression between *B* and *C* is due to load concentration. The sudden increase of subsidence to the left of section *D* can only partially be explained by the larger load. The fact that the sudden drop coincides with the border line of undisturbed and disturbed soil, and not with section *D* where the load increase starts, is a remarkable evidence for the disturbing effect of piles.

However, much is yet to be learned on this phase of the question. There is the possibility that the vibrations produced by wind and other load through buildings and bridges to the footings may produce enough manipulation of the soil immediately beneath the raft footings gradually to soften the soil, so that it would be unsafe to consider the ultimate bearing value much greater than that indicated by the disturbed-soil curve. Also, the disturbances produced by piling might be temporary, so that the foundation set on piles would gradually grow stronger in support.

Actual Settlements Larger than Theoretical.—In addition to the settlements as estimated from compression tests on samples of undisturbed soils, additional settlement also occurs. (This is illustrated in Fig. 97.)

Curve *a* represents the estimate of settlement based on compression-test data, curve *b* the additional settlement, making

the total settlement-time relation that shown by curve *c*, which conforms more closely to those observed in practice and illustrated in Figs. 78 to 82 inclusive.

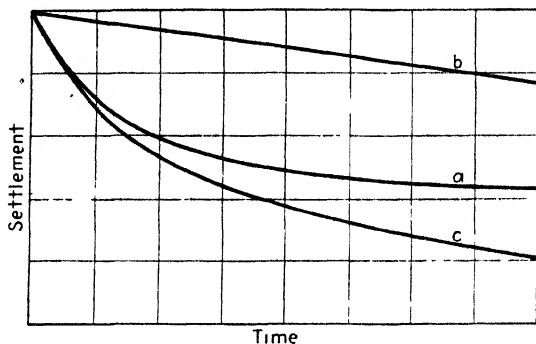


FIG. 97.—Theoretical and actual time-settlement curves compared.

The settlement shown as *b* has been attributed to lateral bulging of the undersoil due to plastic flow.

It has also been explained (113) that in addition to the settlement due to escape of water vertically, water may also escape

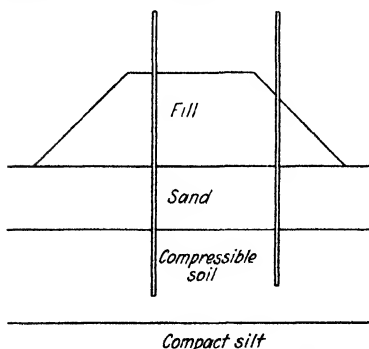


FIG. 98.—Arrangement of pipes for indicating hydrostatic pressures in soil under fill.

laterally. During the period in which the soil is consolidating and prior to the time when equilibrium occurs and the soil skeleton takes the entire load, the hydrostatic pressure under the loaded area is greater than at adjacent locations. Flow of water laterally from under the loaded area to equalize the differences in hydrostatic pressures is the logical result which may be expected.

That the differences in pressure exist has been disclosed by experiments performed in Sweden by placing pipes during construction of a fill as shown in Fig. 98. Immediately after the fill was placed, the water had risen in the tubes to a height indicating a hydrostatic pressure equivalent to the weight of the fill material. The elevation of the water in the pressure-indicating pipes

gradually dropped as the consolidation of the compressible soil layer progressed.

Another contributing factor may be the effect of vibrations in buildings and other structures, mentioned above, for gradually changing the compressibility of the foundation soil from that of the undisturbed to that of the remolded state. In such case the ultimate settlement would be indicated by data furnished by tests on the remolded samples. The rate at which the settlements exceed those indicated by compressibility of the undisturbed samples would then depend upon such factors as the wind and the proximity of highways, railroads, and streetcar lines.

Problems

1. How are the coefficients of compressibility a and consolidation c related?

2. (a) How would the natural moisture contents of old lake and river-bottom materials compare with the liquid limits of these materials?

(b) What factors control the degree of consolidation of soils?

3. In the expression $w = B - Z \log p$ how may the constants B and Z be evaluated?

4. What is the force of capillarity corresponding to a pore width of 0.000023 cm. in the case of the muck soil described on page 183?

5. Given that $k = 1.7 \times 10^{-6}$ cm. per minute, $e_1 = 2.50$, $e_2 = 1.25$, and $a = 0.0002$, find the theoretical coefficient of permeability k_0 and the coefficient of consolidation c .

6. The initial thickness of a laboratory-soil sample at its liquid limit and prior to compression between two permeable stones is 0.3 cm., and the voids ratio is 3.14. After compression for 10 min. the voids ratio becomes 1.96. How long would it take a stratum of the soil 12 ft. thick to be consolidated to the same extent, the initial voids ratio and applied load being the same in the two cases but with the soil stratum covered above with a thick layer of sand and gravel and resting on an impervious-rock base?

7. Neglecting all the terms after the third in equation (46) for the reduction in pore ratio q with uniform pressure distribution, and taking $J_1 = 3.671 t$, find q for t equal to 1 year.

8. (a) Explain the effect of natural structure of a soil on its rate and extent of compression.

(b) How would you expect the degree to which a moist clay is disturbed to vary with the following procedures?

First, pile driving is continuous.

Second, there are 12 hr. intermissions between short periods of driving.

CHAPTER X

SHEAR STRENGTH AND STABILITY

Improvement and extension of theories originally developed by Coulomb, Rankine, and Weyrauch to explain the mechanics of landslides, failure of retaining walls and earth embankments, and the settlement of structures not due to compression of the undersoil have furnished a most fascinating field of endeavor for mathematically minded analysts. Attempts to apply such modifications have been the cause of much controversy. Generally, shear-strength data are more laborious to obtain in the laboratory and can be used less confidently, owing to the number of assumptions involved, than data furnished by the compression tests.

Knowledge of the manner in which the different elements are generally considered to influence the stability of soil is of value qualitatively, not quantitatively, in the design and construction of earthworks.

Mechanics of Lateral Flow.—Lateral flow is the essential characteristic of all landslides, of which there may be two types: detritus and shear.

Detritus slides always occur in soils in their natural position, such as in the side slopes of cuts. These slides move along lubricated surfaces of undersoils, especially on impervious soil layers, which usually provide potential lubricating surfaces.

Shear slides always develop along cylindrical surfaces in cohesive soil layers composed of homogeneous, or nearly homogeneous, material. Fills are particularly susceptible, but these slides are likely to occur also in natural soils in the side slopes of cuts.

Movements of soils due to shear are responsible for the failure of fills due to sliding, the penetration of subgrade soil into the interstices of porous base courses, and the rutting of subgrades and base courses (see Fig. 99).

Soil movement during loss of stability or rutting is illustrated by Fig. 100. It is assumed that the load is applied for an

indefinite length over a width of $2b$. For deformation under load to occur, the section A must shear along some plane such as S and displace laterally, as indicated in Fig. 100b. But for this to occur, the adjacent section marked C must shear along some

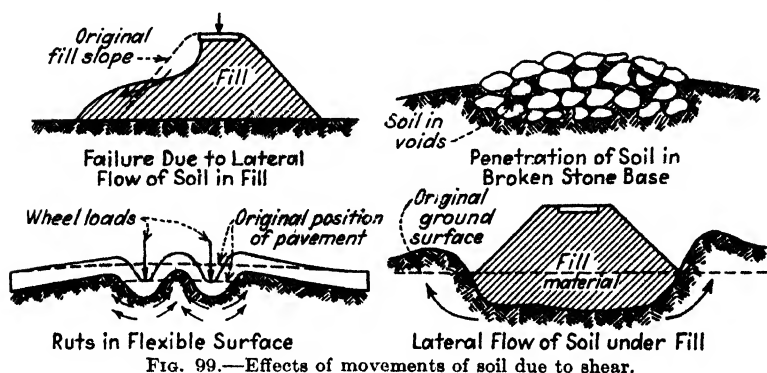


FIG. 99.—Effects of movements of soil due to shear.

surface as S' and, in consequence, displace upward, forming a bulge adjacent to the loaded area as shown in Fig. 100c.

Actually, the surfaces S and S' may be parts of a continuously curved surface, but, for mathematical treatment, they may be

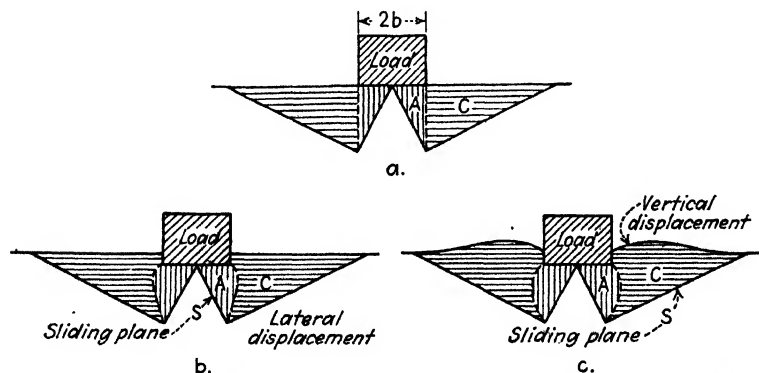


FIG. 100.—Illustrating shear planes along which lateral displacement of soils occurs.

considered as separate plane surfaces without introducing a large error.

If rutting is to be prevented, the prism C must resist displacement sufficiently to prevent the lateral bulging of the prism A .

This may be accomplished by two means, separately or in some combination. Either the shear resistance along the planes S and S' must be sufficiently high to prevent sliding of the prisms, or sufficient pavement weight must be placed adjacent to the loaded area to prevent the upward bulging of the prism C .

The internal friction component of shear strength is furnished by materials such as sand, gravel, slag, or crushed stone, which usually comprise the granular aggregate.

Cohesion is furnished by crystalline chemical formations in Portland-cement mixtures; by bitumen in tar and asphaltic mixtures; by rock or slag powder and moisture films in crushed stone, slag, and gravel mixtures; by colloidal clays and moisture films in topsoil, sand clay, and similar light-textured soils; and by moisture films alone in beach sands stabilized temporarily by the tides.

The basic theory is the same, whether the mixture of granular material and binder becomes a stabilized subgrade, a soil road surface, or a high-type pavement. Only the grading of the granular material and the methods of construction differ.

Binder applied to aggregate already placed produces cement-bound macadam when the binder is Portland cement; bituminous macadam when the binder is tar or asphalt; water-bound macadam when the binder is rock or slag powder; and granular material stabilized with clay when the binder is soil.

Aggregate applied to binder already placed produces the old German sandwich type of concrete pavement, bituminous surface treatment, upside-down macadam, or clay stabilized with granular material, depending upon the kind of binder used.

Aggregate and binder mixed before placement on the roadbed become cement concretes, bituminous concretes, traffic-bound slag and crushed-stone surfaces, or sand clay and sand-clay gravels, depending on the kind of binder used.

The effect of the granular material may be illustrated by two pieces of sandpaper when pressed together. They will exert no resistance to being pulled apart but will resist efforts to slide one over the other, this resistance increasing as the force with which they are pressed together increases.

The effect of binders is illustrated by the performance of two sheets of sticky flypaper when pressed together. Under these conditions they exert high resistance to being pulled apart owing

to the cohesion of the gluelike materials brought into contact with each other.

How the interlocking of the granular particles and the cohesion of the binder combine to furnish shear strength is illustrated in Fig. 101.

A unit shear strength s acting along the sliding plane holds the soil (weight W) in equilibrium and prevents it from sliding.

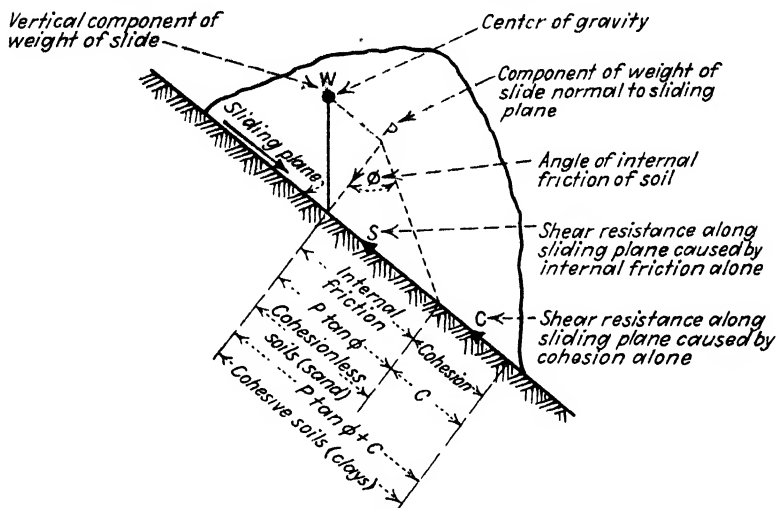


FIG. 101.—Illustrating combined effect of internal friction and cohesion.

That part of the shear strength furnished by the internal friction, or the sandpaper effect, becomes

$$p \tan \phi \text{ per unit of area}$$

in which p = component of W normal to the sliding plane in tons per square foot, kilograms per square centimeter, etc.

ϕ = angle of internal friction: that angle whose tangent equals s/p .

($\tan \phi$ = coefficient of internal friction.)

The cohesion, or flypaper effect, c equals the shear strength when $p = 0$ per unit of area.

The unit shear strength then becomes

$$s = p \tan \phi + c \quad (57)$$

If the material is cohesive, and ϕ is very small, s is very nearly equal to c . According to existing data, however, ϕ is rarely equal to zero.

The total shear strength along the sliding plane is

$$S = P \tan \phi + C$$

Effect of Pressure on Internal Friction.—A block of stone, 1 ft. cube, on an iron surface (Fig. 102A) may be used to illustrate the effect of internal friction. So long as the surface remains horizontal, the entire weight of the block acts perpendicular to the surface to prevent the block from sliding.

Here the shear resistance S corresponding to the shear strength of soils mentioned above becomes Wf , in which f is the coefficient

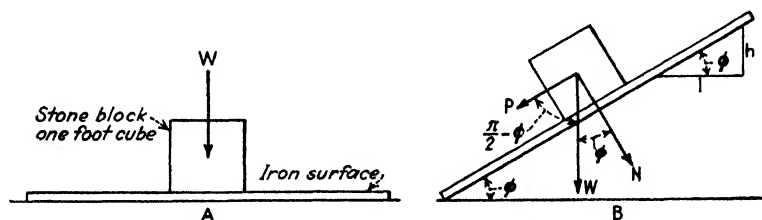


FIG. 102.—Diagram illustrating effect of internal friction.

of friction, corresponding to the angle of internal friction of soils also mentioned above.

When the surface is tilted, as in Fig. 102B, one component N of the weight of the block acts perpendicular to the surface to resist, and another component P acts parallel to the surface to cause sliding of the block. At any slope of the surface, indicated by a vertical rise h in a horizontal distance of unity (Fig. 102B),

$$N = W \cos \phi = \frac{1}{\sqrt{1 + h^2}} W \quad (58)$$

$$P = W \sin \phi = \frac{h}{\sqrt{1 + h^2}} W \quad (59)$$

The force S corresponding to the shear strength mentioned above but that resists sliding in this case equals

$$Nf = Wf \frac{1}{\sqrt{1 + h^2}} \quad (60)$$

As the slope of the iron surface increases, the force productive of sliding increases, and the resistance to sliding decreases until finally a value of h is reached at which P becomes equal to S , and the block begins to slide. This value of h is the coefficient of friction f and also the tangent of the angle ϕ which the inclined plane makes with the horizontal at this time.

According to the Smithsonian tables (86) the stone block should begin to slide when the iron surface reaches a slope designated by an h of about 0.30 (ϕ about 17 deg.). At an average density of 2.70 for the rock, W becomes 168.5 lb., and with the iron plane horizontal

$$S = 0.30 \times 168.5 = 50.6 \text{ lb.}$$

If the stone block were reduced in height until its weight equaled that of a cubic foot of oak, say 44 lb., S would equal

$$0.30 \times 44 = 13.2 \text{ lb.}$$

If, on the other hand, the stone block were loaded until its weight becomes equal to that of a cubic foot of steel, say 475 lb., S would equal about 143 lb.

Angle of Friction Increases with Increase in the Roughness of the Sliding Plane.—If the weight of the block (Fig. 102) is kept constant at 168.5 lb., and the iron surface roughened, the slope at which sliding occurs may be increased until h equals as much as 0.70 (ϕ about 35 deg.). On the other hand, lubricating the surfaces could cause the block to slide on a slope with h as small as 0.10 (ϕ about 6 deg.). In the first case S becomes 118 lb.; and in the second, 17 lb.

Thus the force required to slide blocks having the same coefficient of friction f varies with their weight. That required to slide blocks of the same weight varies with the slope of the inclined plane at which the blocks begin to slide.

Like the resistance of the blocks to sliding, the stability of cohesionless granular materials depends upon their weight and the roughness of their sliding surface. In such soils the entire shear strength along the sliding plane (Fig. 100) is furnished by internal friction. ¹⁴

Theory presented by Terzaghi (26) suggests that in such cases the unit load p which the soil will just support without displacing laterally, the unit weight of the soil W , the width of the loaded

area $2b$, and the angle of internal friction ϕ are mathematically related as follows:

$$p = \frac{bW(1 - \tan^4 \beta)}{2 \tan^5 \beta} \quad (61)$$

in which

$$\beta = 45^\circ - \frac{\phi}{2}$$

According to Eq. (61), the stability varies directly as the width of the loaded area and the weight of the soil. Since the pressure

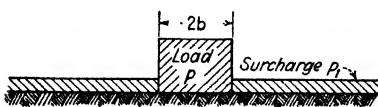


FIG. 103.—Diagram for analysis of effect of surcharge adjacent to load.

of the prism of earth C upon the sliding plane S' is a controlling factor of stability, increasing this pressure by means of a surcharge p_1 on top of this prism, as shown in Fig.

103, serves in turn to increase the stability p . This increase is designated by a new term which causes the formula now to become

$$p = \frac{bW(1 - \tan^4 \beta)}{2 \tan^5 \beta} + \frac{p_1}{\tan^4 \beta} \quad (62)$$

Effect of Soil-particle Weight.—In soils the pressure on the granular fraction must be furnished by the weight of the soil particles. It can readily be seen that no sandpaper effect would exist if the soil particles were without weight. Therefore, the sandpaper effect, and consequently that part of the stability which depends upon the interlocking of grains, increases with the weight of the particles, the degree of surface roughness remaining constant.

Thus one quickly sinks into materials like oats because (a) the grains are very light in weight, (b) the grains are very smooth, and (c) there is no cohesion, or flypaper effect, to hold the grains together.

Similarly, the beach sand referred to in a previous chapter as being highly stable when damp loses an appreciable amount of supporting value when total immersion reduces the effective specific gravity by about 40 per cent (from about 2.65 to 1.65 on the average) owing to the buoyant effect of the water. When water flows upward through the sand, the effective weight of the

particles is still further reduced. It is this loss in effective weight that accounts for the quicksand previously described.

If the material has no cohesion whatsoever, the angle of internal friction depends largely on the density of the material. For high densities the value of ϕ decreases as the pressure is decreased, while for material of low density the effect of pressure on the angle of internal friction is very small. Furthermore, for a perfectly cohesionless material, the height of a given slope has nothing to do with its stability, a fact markedly in contrast to conditions obtaining if the material is cohesive.

The Cohesion Component of Shear Strength.—To illustrate the manner in which cohesion supplements internal friction in furnishing stability, reference is again made to the stone block on the iron surface (Fig. 102).

Assume that the block that slides when h equals 0.30 is to be kept from sliding until h exceeds 1.42 by means of glue applied to the iron base. The strength of this glue, or cohesion, is computed as follows:

The total force P productive of sliding is

$$168.5 \times \frac{1.42}{\sqrt{1 + 1.42^2}} = 0.817 \times 168.5 = 137.7 \text{ lb.}$$

The frictional resistance preventive of sliding is

$$168.5 \times \frac{1}{\sqrt{1 + 1.42^2}} \times 0.3 = 29.1 \text{ lb.}$$

The difference, or

$$137.7 - 29.1 = 108.6 \text{ lb. per square foot}$$

is the required cohesion.

The amount of cohesion c required to increase the angle of repose of soils beyond the angle of internal friction ϕ and the stability of soils above that furnished by the granular particles alone can also be determined. The latter is accomplished by means of a third term added to the stability formula, which now becomes

$$p = \frac{bW(1 - \tan^4 \beta)}{2 \tan^5 \beta} + \frac{p_1}{\tan^4 \beta} + \frac{2c}{\tan \beta \sin^2 \beta} \quad (63)$$

where p = "ultimate bearing capacity."

Shear-strength Determinations.—The essential feature of a test for internal friction is the measurement of the lateral pressure produced by loading a soil with a vertical load. This can be done by means of the improved compression-test apparatus of the U. S. Bureau of Public Roads shown in Fig. 104.

Reducing the compression on the calibrated spring which holds the plunger against the side of the soil cake, by means of the slow-motion screw, until the spring force just balances the first tend-

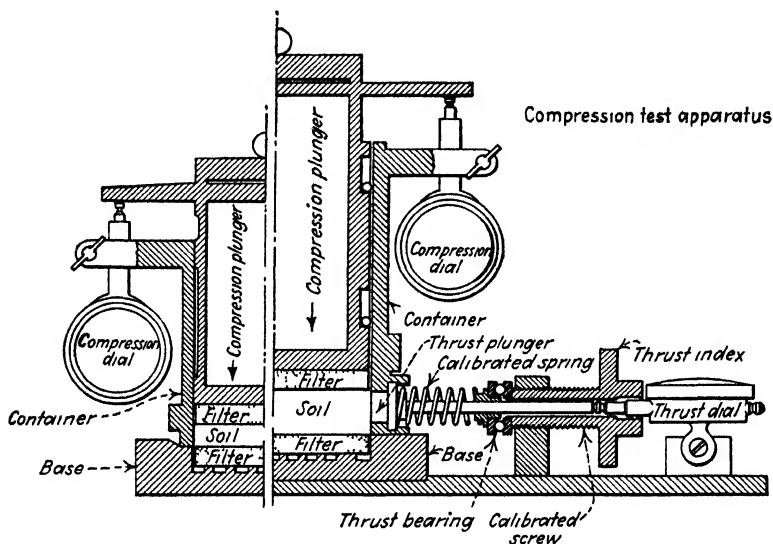


FIG 104 —Compression-test apparatus: original design, left, modified design, right

ency of the plunger to move, discloses the lateral thrust with a minimum bulging of the soil cake.

The theory for determining the angle of internal friction suggested by Terzaghi (90) has been verified by Cox (114). Terzaghi's formula from Mohr's (115) circle of stress is

$$\frac{p_m}{p} = \tan^2 \left(45^\circ - \frac{\phi}{2} \right) \quad (64)$$

where p_m = lateral pressure.

p = applied vertical pressure.

ϕ = angle of internal friction.

Another formula from Mohr's circle, suggested by H. E. Gruner (116), is

$$\tan \phi = \frac{1 - \frac{p_m}{p}}{2\sqrt{p_m/p}} \quad (65)$$

In the test of a sand sample at the laboratory of the Bureau of Public Roads a horizontal pressure of 0.115 ton per square foot was produced by a vertical pressure of 0.33 ton per square foot. By either of the formulas (64) and (65), $\phi = 28^\circ 54'$.

Figure 105 shows the essential features of an apparatus for measuring the shear strength of saturated-soil samples. As in the compression test, the sample is confined between two porous

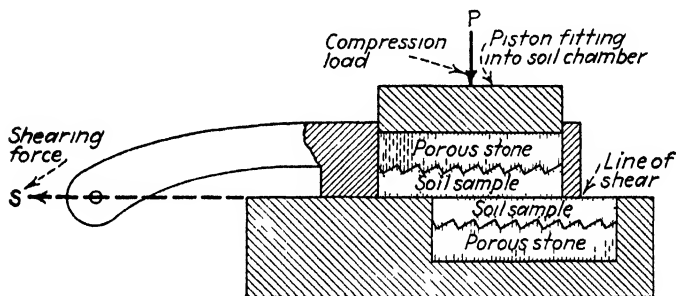


FIG. 105 — Device for measuring shear

stones. The force S required to shear the sample is determined for different values of the vertical load P .

The relationship between S and P for a sandy soil is shown by the broken line in Fig. 106. For sandy soils this line is usually a slight curve for which a straight line can be substituted as shown by the full line (Fig. 106). In sands or other soils having little or no cohesion the shear resistance for practical purposes becomes

$$S = P \tan \phi$$

where ϕ = angle between substituted straight line and horizontal (Fig. 106).

Determinations of the shear strength of cohesive soils require a more involved procedure. In this case the unit shear strength becomes

$$s = p \tan \phi + c \quad (57)$$

When $p = 0$, the shear strength, as noted before, equals the cohesion. While cohesion is independent of the pressure p , it depends to a large extent upon the degree of consolidation of the soil at the time of test.

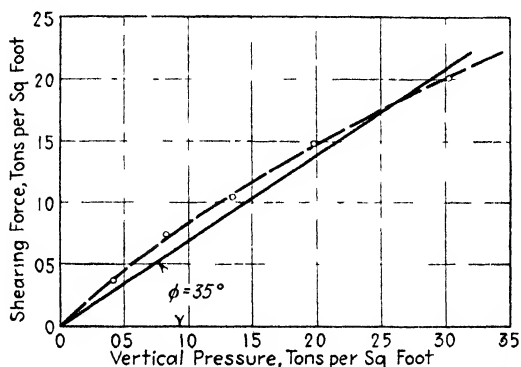


FIG. 106.—Typical shear-test curve for a sandy soil

In the shear test of a cohesive soil, several sets of samples of the soil are compressed to different degrees. Thus four sets of samples of the soil referred to in Fig. 107 were compressed under respective pressures of 1, 2, 3, and 4 tons per square foot. For

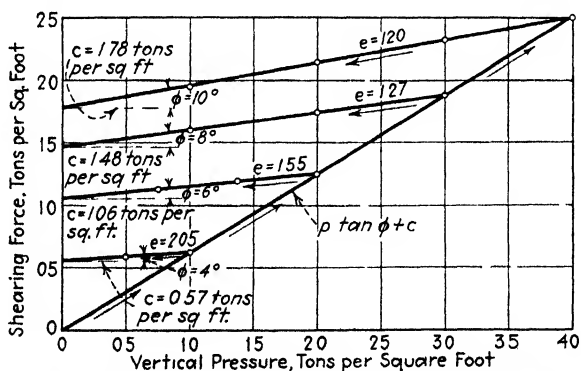


FIG. 107.—Typical shear-test curve for a cohesive soil

each set the shear strength S is determined for a maximum pressure and also for one or more pressures reduced below the maximum. Each value of S thus determined is plotted against the corresponding value of p . The value of S for a pressure of 0, as

indicated by a line passing through the points so plotted, indicates that portion of the total shear strength due to cohesion. It will be noted that the descent from the maximum pressure is along a straight line which deviates considerably from the ascending line which begins at the origin and ends at the maximum pressure. The lines drawn through points plotted when pressures are reduced intercept the ordinate at points above the origin as indicated in Fig. 107.

Thus, according to Fig. 107, the shear strength of the soil under a pressure of 4 tons per square foot was 2.5 tons per square foot. When the load was reduced from 4 to 3 tons per square foot, the shear strength dropped to 2.33 tons per square foot. With reduction of the load to 2 tons and 1 ton per square foot, the shear strength dropped to 2.15 and 1.95 tons per square foot, respectively. The intersection of the line through the points with the ordinate of zero pressure indicates a shearing force of 1.78 tons per square foot, which is therefore the cohesion.

The values of c and ϕ (Fig. 107), it should be emphasized, represent only approximations. The $\tan \phi$ equals the coefficient of friction only of sands. For clays the line connecting the " $p \tan \phi + c$ " curve with the 0 load ordinate is not straight but is curved. The true angle of internal friction may be less than that indicated by the procedure (Fig. 107). For the purposes of this text, however, more than simply to mention that differences do exist between the true and approximate values is unnecessary.

Stability Represents the Combined Effect of a Number of Agencies.—By substitution of numerical values in stability formula (63) some conception of the relative influence of the more important road-surface variables can be obtained. The approximate results of such computations are shown in Table 22.

For the conditions designated in this table, the effect of increasing ϕ from 10 (that of the silt) to 34° (that of the sand) is to increase the stability per square foot from 15 to 270 lb. The effect of immersing the dry sand and thereby reducing its effective weight per cubic foot from 100 to 63 lb. is to reduce the stability from 270 to 170 lb.

Increasing the width of load distribution on the cohesionless sand ten times increases the stability from 270 to 2,710 lb. per square foot. Surcharging the sand adjacent to the load with 100 lb. per square foot, which is approximately equivalent to the

TABLE 22.—INFLUENCE OF INTERNAL FRICTION, COHESION, WIDTH OF LOADED AREA, AND LOAD ADJACENT TO THE LOADED AREA UPON THE STABILITY OF SOILS

Soil types	Cohesion c , pounds per square foot	Angle of internal friction ϕ , degrees	Supporting value pounds per square foot			Cohesion c , pounds per square foot, to compensate for	
			$p_1 = 0$ $S = 100$ $b = 0.25$ foot	$p_1 = 100$ $S = 100$ $b = 0.25$ foot	$p_1 = 0$ $S = 100$ $b = 2.5$ feet	$p_1 = 100$	$b = 2.5$
Silts, wet.....	0	10	15	220	150	35	25
Sands, dry.....	0	34	270	1,520	2,710	75	145
Immersed ($S = 63$).....	0	34	170	1,420	1,710	75	90
Clay, liquid.....	100	0	400	500	400		
Very soft.....	200	2	860	980	880		
Soft.....	400	4	1,850	1,980	1,890		
Fairly stiff.....	1,000	6	4,970	5,120	5,030		
Very stiff.....	2,000	12	12,490	12,720	12,680		
Cemented sand and gravel, wet.....	500	34	8,800	10,060	11,240		
Cemented sand and gravel.....	1,000	34	17,340	18,590	19,770		

weight of a pavement 8 in. thick, increases the stability from 270 to 1,520 lb. per square foot.

The stability of clays may be increased enormously by reducing their moisture contents or by using admixtures of sand; and that of sand may be increased by admixtures of clay.

When the cohesion is 1,000 lb. per square foot, b is 0.25 ft., and p_1 is 0, the supporting value p will equal either 4,970 or 17,340 lb. per square foot, depending on whether ϕ equals 6 or 34 deg. When ϕ equals 34 deg., p will equal 270, 8,800, or 17,340 lb. per square foot, depending on whether c equals 0, 500, or 1,000 lb. per square foot.

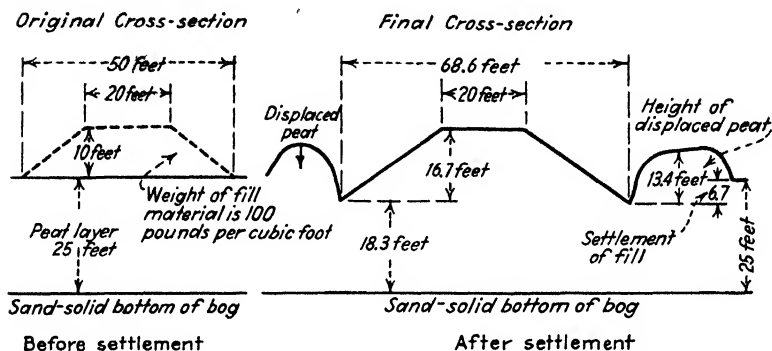


FIG. 108.—Diagram for computing settlement of a fill due to displacement of soft undersoil.

It is especially interesting to note from Table 22 the small amounts of cohesion required to increase the stability of cohesionless materials considerably. In the silt with ϕ equal to 10 deg., a cohesion c equal to but 431 lb. per square foot is needed to raise p from 15 to 2,500 lb.; and c equal to but 131 lb. is needed to raise p of the sand from 270 to 2,500 lb. per square foot.

Fill Foundations.—Some conception of the stability of soft undersoils also may be obtained by means of Fig. 108 and formula (61) in which

p = ultimate bearing value of peat bog, pounds per square inch.

p_1 = ultimate pressure of displaced peat at sides of fill, pounds per square inch.

b = one-half effective width of fill = $10 + \frac{1}{2}$ slope = 17.5 ft.

W = weight of peat, pounds per cubic foot = 80.

c = cohesion of the peat, pounds per square foot, in this case considered as zero.

$$\beta = 45^\circ - \frac{\phi}{2} = 43^\circ.$$

(ϕ = angle of internal friction of peat = 4 deg.).

Assuming that the 10-ft. fill is completed before displacement of the peat begins, $p_1 = 0$; and since $c = 0$,

$$\begin{aligned} p &= \frac{bW(1 - \tan^4 \beta)}{2 \tan^5 \beta} \\ &= \frac{17.5 \times 80(1 - \tan^4 43^\circ)}{2 \tan^5 43^\circ} = 240 \text{ lb. per square foot} \end{aligned} \quad (61)$$

The pressure of the 10-ft. depth of fill upon the peat (assuming a weight of 100 lb. per cubic foot) is 1,000 lb. per square foot. The weight p_1 of the displaced peat necessary to counterbalance the weight of the fill so as to stop the settlement is obtained as follows:

$$\frac{p_1}{\tan^4 43^\circ} = p - 240 = 1,000 - 240 = 760 \text{ lb. per square foot}$$

Then

$$p_1 = 760(\tan^4 43^\circ) = 575 \text{ lb. per square foot}$$

Since the weight of the displaced peat is 80 lb. per cubic foot, the height of the displaced peat above the bottom of the fill is $\frac{575}{80} = 7.2$ ft., and the settlement of the fill is one-half this amount, or 3.6 ft.

Additional material must then be added to the fill from time to time until ultimate settlement occurs. Let x = total depth of additional material required to bring the fill to ultimate grade. For each foot of material added, the settlement of the fill is $\frac{3.6 \times 100}{760} = 0.47$ ft. Then $x - 0.47x = 3.6$ and $x = 6.7$ ft.

Thus the total depth of the entire fill when ultimate settlement has occurred is $10 + 6.7 = 16.7$ ft., and the height of the displaced peat above the bottom of the fill is $6.7 \times 2 = 13.4$ ft.

Critical Height of Fills.—In estimating the critical height of fills two cases are involved: (a) cohesionless soils and (b) cohesive soils (see Fig. 109). In sands the angle of the side slope θ , the

angle of internal friction ϕ , and the angle of repose may all be considered equal. Fills constructed of such materials have no critical height.

In cohesive soils the angle of repose (safe slope) varies with the height of the fill and has no relation to the angle of internal friction ϕ .

Coulomb's formula ($s = p \tan \phi + c$) assumes a plane sliding surface for cohesive soils, instead of the cylindrical surface which involves intricate mathematical computations.

Résal's formula (117), which is believed to give a closer approximation of the true critical height for cohesive soils, although it also assumes a plane sliding surface, is as follows:

$$H = \frac{c \sin \theta \cos \phi}{W \sin^2 \frac{\theta - \phi}{2}} \quad (66)$$

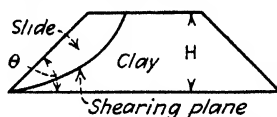


FIG. 109.—Diagram illustrating critical height of fill.

where H = critical height above which sliding will occur, feet.

W = weight of fill soil, pounds per cubic foot.

Later, Frontard extended and revised Résal's formula. According to Frontard, the sliding curve is a deformed cycloid, approaching very nearly a circle. None of these formulas has general application. The graphical method of Petterson, Krey, Hultin, Fellenius, and others is based essentially on the assumption that the sliding surface is cylindrical. Observations where slides have occurred indicate the reasonableness of this assumption. The relation between the radius of curvature, the position of the center of the sliding circle, the inclination of the slope, and the coefficient of internal friction are used in the graphical method which is coming to be accepted by leading authorities. The essential features of this method have been summarized by Charles Terzaghi in *Public Roads* (118).

Critical heights for various slopes in cuts and fills as computed by Résal's formula and by the foregoing graphical method are shown in Table 23.

Effect of Undersoil on Fill Slopes.—The safe slope of an embankment that is to be placed on a soft undersoil may be computed from the formula

$$b = \frac{pd}{\sigma} \quad (67)$$

where b = one-half base width of embankment.

p = unit pressure produced by embankment.

d = one-half depth of soft undersoil layer.

s = shearing strength of undersoil.

This formula is based on the assumption that the external pressure on the undersoil is produced by an embankment having a

TABLE 23.—CRITICAL HEIGHTS OF SLOPE IN CUTS AND FILLS¹

Kind of soil	Slope of cut or fill	Angle of slope θ	Weight of soil W , lb. per cu. ft.	Cohesion of soil c , lb. per sq. ft.	Angle of internal friction ϕ , degrees	Critical height of fill, H , feet	
						2	3
Very soft clay	$\frac{1}{2}:1$	63 26	80	200	2	13	9
	1:1	45 0				16	13
	2:1	26 34				18	25
	4:1	14 02				23	55
Medium clay	$\frac{1}{2}:1$	63 26	90	1,000	6	67	43
	1:1	45 0				84	70
	2:1	26 34				113	155
	4:1	14 02				198	546
Stiff clay	$\frac{1}{2}:1$	63 26	100	1,500	8	95	61
	1:1	45 0				125	104
	2:1	26 34				180	255
	4:1	14 02				403	1,300
Good sand clays	$\frac{1}{2}:1$	63 26	110	1,000	34	162	104
	1:1	45 0				468	580
	2:1	26 34				Unlimited	Unlimited
	4:1	14 02				Unlimited	Unlimited
Silty clays	$\frac{1}{2}:1$	63 26	100	200	14	15	10
	1:1	45 0				22	20
	2:1	26 34				43	72
	4:1	14 02				Unlimited	Unlimited

NOTE.—Materials possessing no cohesion, such as sands, silts, etc., have no critical heights.

¹ All computations made by F. A. Robeson.

² Computations by Réaumur's formula.

³ Computations by the graphical method of K. E. Petterson as revised and extended by Sven Hultin, W. Fellenius, H. Krey, Charles Terzaghi, and others.

triangular cross section. In practice embankments are trapezoidal in shape. In order to estimate p , it is necessary to make an approximation. The height of the embankment is considered as increased by an amount equal to b_c/l where b_c is one-half the width of the crest of the embankment, and l is the horizontal distance corresponding to a vertical rise of 1 ft. for an assumed safe side slope. The value of p is then obtained by multiplying the weight per cubic foot of the embankment material by the increased height of embankment.

The value of d is determined by borings. Shear tests on undisturbed samples furnish the value of s .

The value of b is then computed. This is the minimum value of b for the height used at which the foundation is safe against failure by shear.

Tests of Soil in Place.—Tests to determine the direct bearing value of foundation soils are generally conducted by excavating a test pit down to the elevation on which the foundation is to rest and measuring the settlement due to loads placed on a platform supported on a post through which the load is transferred to a bearing plate at the foot of the post. The method of applying the load and measuring the deflection may vary with the individual ideas of the engineer in charge of the work.

The apparatus proposed by the Specification Committee of the American Society of Civil Engineers to Codify Practice on the Bearing Value of Soils for Foundations of the A.S.C.E. is shown in Fig. 110 (119).

Figure 111 is a typical load-deflection curve for a bearing-power test (120). The portion ab represents the initial compression caused by a recompression of the topsoil layer which has been loosened in excavating the pit, plus the elastic yielding for that load increment. The portion bc represents the true elastic curve of the soil, and the portion cd the breaking down after the elastic limit is passed. The soil tests should be carried far enough to develop the full curve for each test, after which the load above the point c should be removed, and the remaining load left in place for at least four additional days during which time the settlement should be carefully observed. If the continued application of this load without increment causes an increased deflection, the test should be repeated using a lower position for the yield point. This should be repeated if necessary until there is no further

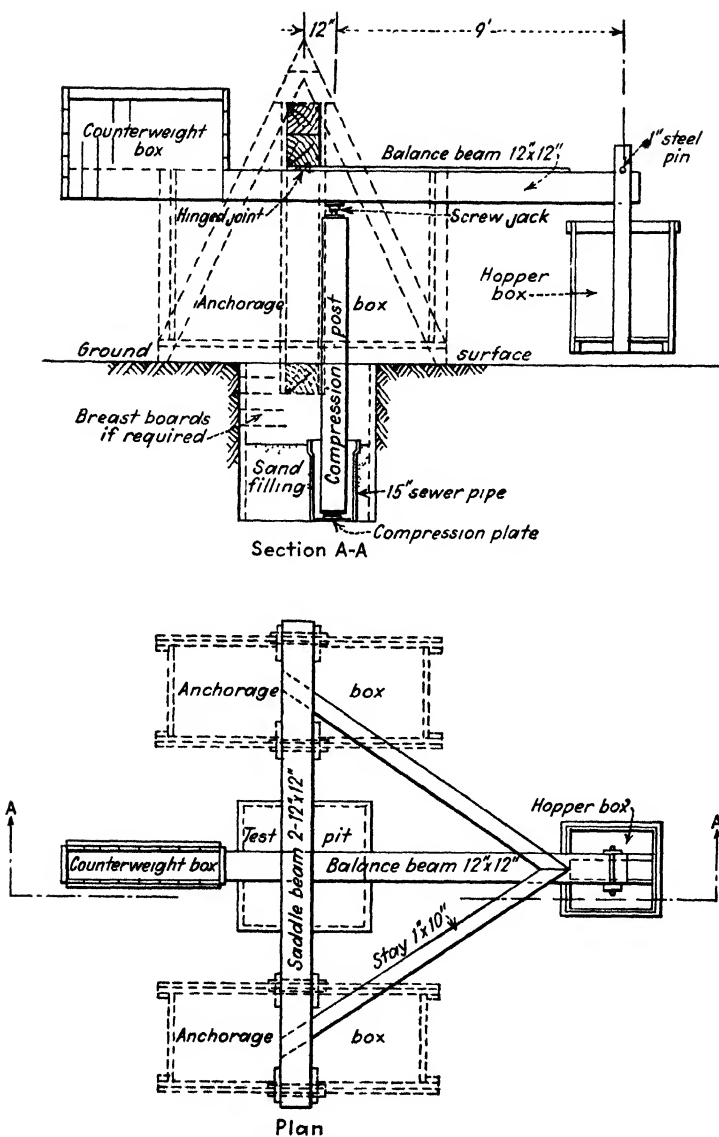


FIG. 110.—Load-testing apparatus for soils.

yielding after 4 days' continued application of load. The point thus found is the elastic limit of the soil in compression (120).

The safe bearing value of the soil is generally limited to one-half of the unit load at the elastic limit.

Perimeter-shear Theory.—Based on loading tests performed on soils in place, with bearing areas differing in size and shape, Prof. W. S. Housel of the University of Michigan developed a method for the selection of foundations which involves the shear along the perimeter of bearing plates, as well as their areas. The essential features of this method of determining bearing capacity are disclosed by the following summarization obtained from *Engineering Research Bulletin 13*, University of Michigan (19),

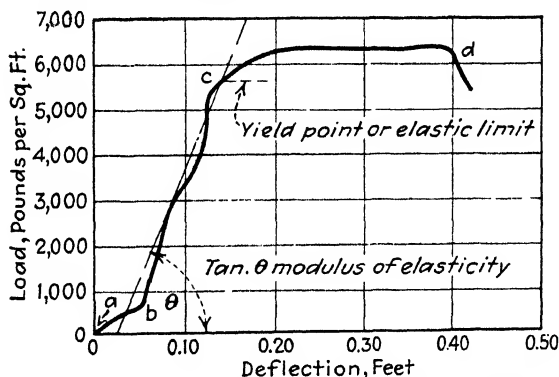


FIG. 111.—Typical load-deflection curve for foundation bearing-power test.

and a discussion from the *Transactions of the American Society of Civil Engineers*, 1929 (108).

The theory of soil settlement advanced by Professor Housel is based on the results furnished by loading tests made in 1927 in connection with a grade-separation project located in the River Rouge valley at the intersection of Fort Street with the Pere Marquette, Wabash, and Michigan Central Railways, Wayne County, Mich. The laboratory facilities at the University of Michigan were made available through the courtesy of the State Highway Department in conducting the investigation, with further assistance rendered by the Department of Engineering Research of the university.

Additional investigation in the field was carried out during the summer of 1928. The loading apparatus used was modeled

after that recommended by the Specifications Committee of the American Society of Civil Engineers and shown in Fig. 110. It had a loading capacity of 50 tons. Both round and square bearing plates were used with four different areas of 1, 2, 4, and 9 sq. ft., respectively.

Investigation included attempts to measure lateral pressure also. The pits in which the bearing plates were placed were spaced 20 ft. center to center in order to insure that the soil in one pit would not be affected by a previous test. The general soil profile at the site of the tests was as follows: A surface layer of 6 to 13 in. of loam and humus and approximately 2 ft. of yellow sand, a stratum of 4 to 5 ft. of stiff yellow clay, a stratum of 4 to 5 ft. of plastic yellow clay, below which was homogeneous, finely divided plastic blue clay which extended to hardpan at a depth of approximately 90 ft.

The tests indicated that there were two factors of strength by virtue of which cohesive soils, such as clay, are capable of supporting loads. One of these factors is the shearing resistance on the perimeter of the bearing area; the other is the resistance of the soil to compression, due to a condition of strain set up in the soil around the bearing area, which has been referred to previously as the pressure bulb.

The first failure during test was noticed to be in shear along the perimeter of the bearing area. Upheaval of the soil surrounding the bearing area, indicative of strain, was noticeable in the latter part of the test. In the first part of the tests (the range of bearing capacity in which shear failure about the perimeter was taking place) the relation between the bearing capacity of the several plates was found similar to the relation between the values of the perimeter-area ratio P'/A , termed x , for the different plates. The bearing capacities were not in exact proportion to x , because even in their lower ranges the pressure bulb was beginning to have some effect as a factor of strength. In the latter part of the tests, where the pressure bulb was indicated to be a controlling factor, there was found to be a complete reversal of bearing-capacity relations, the plates with the largest P'/A values having the smallest bearing capacities.

The equation for bearing capacity found by Housel to be most useful in interpreting test data, and fitting the test data most

satisfactorily, includes both factors of strength and has the form

$$P = Ap = P's + Ap'$$

and

$$p = \frac{P's}{A} + p' \quad (68)$$

where p = bearing capacity, pounds per square foot.

s = shear on perimeter, pounds per linear foot.

p' = resistance of soil to compression, pounds per square foot.

P' = perimeter of area, feet.

A = area of footing, square feet.

P = total allowable load on footing.

By substitution of x for P'/A the expression becomes

$$p = sx + p' \quad (69)$$

Equation (69) is the expression of a linear function involving two variables, *viz.*, bearing capacity and perimeter-area ratio. When two bearing areas, having different perimeter-area ratios, are used in the test, it is possible to solve the two resulting equations for the constants s and p' . The bearing-capacity equation can then be used to determine the allowable unit pressure for larger footings, such that the limiting settlement will not be exceeded.

Results of the tests on the blue clay are given, together with the solution of the equations by the method of least squares, in Table 24. The following points of view were considered worthy of emphasis by Housel (19) in connection with the bearing-capacity equations and curves: (a) As the size of the footing is increased, and x approaches zero, the bearing capacity approaches not zero but a constant value which is the compressive strength of the soil. (b) For a type of soil that has a high degree of fluidity and low cohesive strength, the bearing capacity is more nearly constant for all sizes, indicating that x is the least important factor in the bearing capacity. (c) In practical sizes of footing, having perimeter-area ratios between zero and 1, the shape of the footing, whether round or square, is not so important. This is consistent with the idea that loss in bearing capacity (as controlled by the pressure bulb), due to the sharp corners, is large in proportion to

TABLE 24.—SERIES 2. BLUE CLAY

Test number	Size of area, square feet	Shape	P'/A ratio	Settlement, inches	Bearing-capacity limit, lb. per sq. ft.	
					Tested	Figured
43	1	Round	3.55	0.60	5,700	5,600
39	2	Round	2.51	0.20	4,750	4,910
18	4	Round	1.77	0.60	4,425	4,415
14	9	Round	1.19	0.50	4,100	5,025
41	1	Square	4.00	0.55	5,625	5,320
37	2	Square	2.83	1.2	4,100	4,595
12	9	Square	1.33	0.44	3,850	3,665

the total area of the plates. (d) In a type of soil, such as the blue clay, having a high degree of fluidity the pressure bulb is a controlling factor throughout, and the difference in bearing capacity, due to the shape of the plate, is practically constant.

Computed Bearing Values Compare Favorably with Observed Values.—The increases in the unit support of cohesionless soils due to increase in width of loaded area and to the surcharging just noted are much greater when the loaded area is long and narrow than when it is square or circular (27). Observations for the effect of size of loaded area have not yielded consistent results.

Goldbeck and Bussard (122) found small settlements produced by the same unit load to be related according to the expression

$$\frac{Q}{Q_1} = \frac{A}{A_1} \quad (70)$$

where Q = settlement of area A .

Q_1 = settlement of area A_1 .

A similar finding is reported by Kölger (123) for his tests and also those performed in the field for direct compression by Henrick Press (124) and Housel (125).

Prior (126), however, found that piers settled on yellow clay and blue clay 0.1 and $\frac{5}{16}$ in., respectively, when, according to the Goldbeck formula, they should have settled 1.45 and $5\frac{1}{2}$ in., respectively. As a matter of fact, the settlement of the yellow clay was the same for areas of 0.4 and 85 sq. ft.

Terzaghi (108) dispels the seeming inconsistency of the foregoing data by calling attention to the difference in the settlement properties of cohesive and cohesionless materials. This is discussed also by Besson (127). He suggests that settlements of the former follow those designated by the formula, while the size of the bearing area should have comparatively little effect on settlements of the cohesionless materials.

Prior inclines to a formula for bearing value suggested by C. C. Williams (128) and used also by Housel (125) and just discussed.

Here the presence of shearing capacity causes the supporting power per unit of area to decrease as the area increases. However, as pointed out by Housel (125): "As the size of the footing is increased and the value of P'/A approaches zero, the bearing capacity approaches a constant value p' ; the strength of the pressure bulb and perimeter shear s become negligible, so that the same bearing capacity could be used for different-sized footings without appreciable error." In experiments performed by the Bureau of Public Roads on silt-loam soil it was indicated that s becomes negligible when the loaded area exceeds about 5 sq. ft. Kölger (123) agrees that settlements for large areas are correspondingly smaller than those indicated by the formula. Gil-

TABLE 25.—BEARING VALUES USED AND OBSERVED IN PRACTICE
In pounds per square foot

Material	Digest of build- ing codes	Observed under stable structures		
		Mini- mum	Average	Maxi- mum
Quick sand, alluvial soil.....	1,000	3,000	6,000	12,000
Soft clay.....	2,000			
Moderately dry clay; fine, clean, dry sand.....	4,000	5,000	9,000	12,000
Fine, dry loam and clay; fine sand..	6,000	4,000	10,000	16,000
Compact, coarse sand; stiff gravel...	8,000			
Coarse gravel; stratified stone and clay; soft rock.....	12,000	5,000	10,000	16,000
Gravel and sand well cemented.....	16,000			
Good hardpan or hard shale.....	20,000	6,000	18,000	24,000
Native bedrock.....	40,000			

boy (89) also finds that "settlement under a given load, not too near the ultimate, is practically independent of the size of the loaded area," an idea that represents current practice.

Although purely theoretical, the values in Table 22 represent a compilation of what at present are considered by the authors the most reliable data derived from different sources on bearing values. They check reasonably well with values used in building codes, a digest of which is shown in Table 25. The values in this table, it will be noted, are general and without regard for the effect of surcharge or size of bearing area.

Problems

1. Distinguish between detritus and shear slides.
2. (a) What materials furnish internal friction? Cohesion?
(b) Discuss the principle of applying binder to aggregate already placed and the converse, applying aggregate to binder.
3. Illustrate diagrammatically the combined effect of internal friction and cohesion.
4. For what value of ϕ would the components N and P (Fig. 102B) be equal?
5. In the formula

$$p = \frac{bW(1 - \tan^4 \beta)}{2 \tan^5 \beta} + \frac{p_1}{\tan^4 \beta}$$

(neglecting all other terms) what is the value of ϕ if $b = 1$ ft., $W = 100$ lb. per cubic foot, and p_1 is one-fourth of p , which is 1 ton per square foot?

6. How would you demonstrate the meaning of "quicksand"?
7. Discuss the relative values for ϕ and c in the materials clean sand, silt, and clay. How does it happen that the stability of a mixture of clay and sand is greater than that of either of these materials taken separately?
8. When considering the separate materials sand and clay, what is the meaning of the "angle of repose"?
9. What experimental data are required in computing the safe slope of an embankment to be placed on a soft undersoil?
10. Explain how the modulus of elasticity and the elastic limit of foundation soils may be determined experimentally.

PART IV
PRACTICE IN DESIGN AND CONSTRUCTION

CHAPTER XI

CLASSIFICATION OF SOILS FOR ENGINEERING USES

The grouping of subgrade soils published originally in *Public Roads* (26) has proved of considerable assistance in the reporting of laboratory-test results, the design of drainage and other subgrade treatment, the selection of materials for use in stabilized soil roads, and the formulation of specifications for fill and embankment construction.

Subdivision of the groups as first described was contemplated, but as studies of soil progressed no general use for increasing the number of main divisions was found.

Characterizing Materials Disclosed by Test Data.—The routine tests serve to disclose the presence of constituents which, by surveys, investigations, and experience, have been found to have a characterizing influence upon road construction and maintenance practices. Sand, silt, clay, colloids, kaolin, mica, peat, the flocculating limes and magnesia, and the deflocculating potash and sodas contribute individual properties indicated by test constants related in ratios covering a wide range. The ratios supplement the constants as such, in the identification of properties productive of conspicuous performance.

Materials with the following properties are indicated by results of tests as follows:

Liquid limit.....	Capillary capacity of soil when thoroughly manipulated
Plasticity index.....	Cohesion
Centrifuge moisture equivalent..	Combined effect of capillarity, compressibility, and permeability
Shrinkage limit.....	Combined effect of cohesion and resistance to consolidation
Field moisture equivalent.....	Combined effect of capillarity and cohesion—the moisture capacity of soils not manipulated
Volumetric change and lineal shrinkage.	Combined effect of capillarity, cohesion, and resistance to consolidation

High liquid limits may be caused by capillary moisture, cohesive films, or a combination of both. Cohesionless mica flakes, spongy diatomaceous earth, and colloidal clay could all have a liquid limit, say, of 200. Then the liquid limit of various mixtures of these materials would also be 200. Colloidal clay could be added to cohesionless diatoms in increasing amounts so that every degree of cohesion is represented within the limits of the two materials without change in the liquid limits.

This is not true for the plastic limits. In cohesionless materials there is no plastic limit, which means, at least theoretically, that the plastic limit is equal to the liquid limit.

Colloidal clay added to the diatoms in increasing amounts will cause the plastic limits of the mixtures to decrease until the minimum of possibly 35, the plastic limit of the pure colloidal clay, is reached. Thus for equal liquid limits, the lower the plastic limit the greater is the indication of the presence of cohesive films and of plastic clays which furnish cohesion.

The smaller the cohesion of soils for equal liquid limits the greater is likely to be the elasticity. The shrinkage, like the plastic, limits of such soils nearly equal the liquid limits. As the cohesion increases, the elasticity diminishes, and the shrinkage limits become smaller for equal liquid limits.

The field moisture equivalents like the shrinkage limits of cohesionless soils are likely to equal the liquid limits. The greater the cohesion of the soil the smaller become the field moisture equivalents for equal liquid limits.

Thus, the difference between the liquid limit and the shrinkage limit and the difference between the liquid limit and the field moisture equivalent, like the plasticity index, indicate the cohesive and plastic properties of the soils. The difference between the liquid limit and shrinkage limit also indicates the maximum shrinkage properties of the soils, whereas the difference between the field moisture equivalent and the shrinkage limit is assumed to indicate the relative shrinkage of soils under field conditions.

Effect of Change in Constituents Illustrated.—The properties of relatively inert materials such as ground quartz may be changed by admixtures in increasing amounts to the properties of the more active soil constituents such as the highly plastic colloidal bentonite. This indicates definite relationships between

the test constants and the percentages of the constituents which furnish the characterizing properties of soils (Fig. 112).

The relationships between the liquid limits LL , the plasticity indexes PI , and the percentage of admixtures P of the preceding materials are as follows:

$$\begin{aligned} PI &= 0.21P = 0.19(LL - 18) \text{ diatoms} \\ &= 0.15P = 0.71(LL - 18) \text{ kaolin} \\ &= 3.3P = 1.0(LL - 18) \text{ bentonite} \end{aligned}$$

If any particular amount of cohesion is desired, say that indicated by a plasticity index of 5, the admixture according to these formulas may be 24 per cent of diatoms, 33 per cent of kaolin, or 1.5 per cent of bentonite. The corresponding capillarity will be indicated by a liquid limit of 44 for diatoms, 25 for kaolin, and 23 for bentonite.

Test Constants Vary Widely.

Results of tests on well-known commercial materials, as well as on soils, are given in Table 26.

Generally, the characteristics of these materials include all those found in natural soils. Bentonite, at the top of the list, is the most active of the gluey colloids productive of compressibility, plasticity, and shrinkage of soils. Ground quartz, at the bottom, is representative of the most stable of cohesionless fine-grained materials of more or less constant volume.

Zinc oxide, the finest material in the group, is representative of the relatively inert clays in glacial formations which are responsible for enormous frost heave. Diatoms and mica, with little or no cohesion, high capillarity, and high shrinkage limits, are productive of elasticity of soils. Pumice, with no cohesion and a fairly constant and high moisture content at the several consistencies, is representative of those quicksands which appear dry on the surface but quake and flow at the slightest manipulation. The clay soil is typical of clay soils found in nature. The addi-

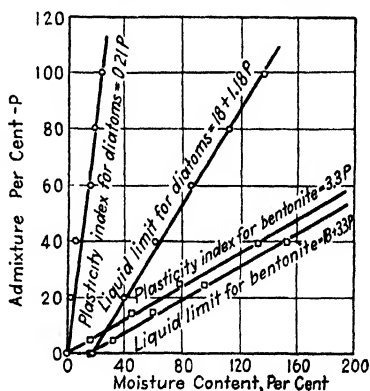


FIG. 112.—Relations of liquid limits and plasticity indexes to percentages of admixtures of diatoms and bentonite with ground quartz.

tion of diatoms and peat to such soils produces materials having the properties of the muck soil.

Kaolin is representative of good pottery clays, which also make excellent binders for topsoil, sand-clay, and gravel-road surfaces. Talc has the physical properties of the natural cohesionless silt soils noted for the occurrence of detrimental frost heave.

TABLE 26.—TEST CONSTANTS OF TYPICAL FINE-GRAINED MATERIALS

Material	Grading		Consistency				Shrinkage		Volume change from liquid limit, per cent	Flow curve			Compression			
	Effective size, millimeters	Uniformity coefficient	Flocculation limit, per cent	Liquid limit, per cent	Plastic limit, per cent	Plasticity index	Limit	Ratio		Flow index <i>F</i> per cent	Intercept <i>H</i> , per cent	Toughness index	Compression index <i>Z</i> , per cent	Intercept <i>B</i> , per cent	Expansion factor	Specific-gravity
Bentonite.....	0.0002	3 2	1,060	399	45	354	6	2 2	865	33.0	438	10.7	190.5	220.8	21.9	2 83
Diatoms.....	0.0003	9.1	337	136	115	21	125	0 6	7	31.7	181	0.7	24.0	159.4	4.3	2 12
Mica.....	0.0023	9.1	214	116	...	0	165 ¹	0.6	0	44.0	176	0	68.1	113.0	13.0	2 85
Muck.....	0.0003	7.0	269	112	39	73	33	1.4	111	28.4	152	2.6	28.4	71.4	3.1	2.71
Zinc oxide.....	0.0001	2.4	408	89	35	54	31	1.9	110	61.0	147	0.9	48.3	68.0	0 25	59
Clay soil.....	0.0002	8 8	236	80	29	51	13	1.9	127	25.0	114	2.0	25.7	54.7	0.9	2.76
Pumice.....	0.0066	5.0	64	52	...	0	57 ¹	1.0	0	4.5	58	0	5.3	52.5	0.4	2.34
Kaolin.....	0.0004	11.0	186	43	28	15	37	1.3	8	14.0	63	1.1	10.3	40.7	1.4	2 63
Talc.....	0.0028	7.5	123	36	25	11	29	1.4	10	18.6	61	0.6	12.3	40.4	1.3	2.77
Quartz flour.....	0.0021	19.1	46	18	...	0	26 ¹	1.6	0	4.9	26	0	1.9	23.1	0 4	2 67

¹ Computed from shrinkage ratio and specific gravity.

TABLE 27.—MATERIALS IN TABLE 26 ARRANGED IN DESCENDING ORDER OF EXTENT OF FINENESS AND OTHER PROPERTIES

Fineness	Flocculation limit	Liquid limit	Plasticity index	Volume change from liquid limit	Flow index <i>F</i>	Compression intercept <i>B</i>	Expansion factor
Zinc oxide.....	Bentonite	Bentonite	Bentonite	Bentonite	Zinc oxide	Bentonite	Bentonite
Bentonite.....	Zinc oxide	Diatoms	Muck	Clay	Mica	Diatoms	Mica
Clay.....	Diatoms	Mica	Zinc oxide	Muck	Bentonite	Mica	Diatoms
Muck.....	Muck	Muck	Clay	Zinc oxide	Diatoms	Muck	Muck
Diatoms.....	Clay	Zinc oxide	Diatoms	Talc	Muck	Zinc oxide	Kaolin
Kaolin.....	Mica	Clay	Kaolin	Kaolin	Clay	Clay	Talc
Quartz flour.....	Kaolin	Pumice	Talc	Diatoms	Talc	Pumice	Clay
Mica.....	Talc	Kaolin	Mica	Mica	Kaolin	Kaolin	Pumice
Talc.....	Pumice	Talc	Pumice	Pumice	Quartz flour	Talc	Quartz flour
Pumice.....	Quartz flour	Quartz flour	Quartz flour	Quartz flour	Pumice	Quartz flour	Zinc oxide

The widely different grain structures of several of the materials shown previously in Fig. 1 adequately explain the difference in performance. Amorphous organic matter and a diatom can be seen in a sample of the muck soil (Fig. 113).

The great range of characteristics of these materials is noted as follows:

1 Maximum moisture content indicated by the flocculation limit, 46 to more than 1,000 per cent.

2 Moisture content at which the materials have equal shear resistance, as indicated by 25 blows signifying the liquid limit, 18 to 399 per cent.

3. Cohesion indicated by the plasticity index, 0 to 354 per cent

4 Shrinkage properties as indicated by the volume change from the liquid limit, 0 to 865 per cent

5 Relative effect of moisture increase for reducing the shear resistance of the soil as indicated by the flow index, 4 to 61 per cent.

6. Moisture content at which the samples in confined state are capable of supporting a load equal to 1 ton per square foot as indicated by the compression intercept *B*, 23 to 221 per cent

7 The relative ability of the materials to absorb water and expand upon the removal of load as indicated by the expansion factor, 0.2 to 21.9 per cent.

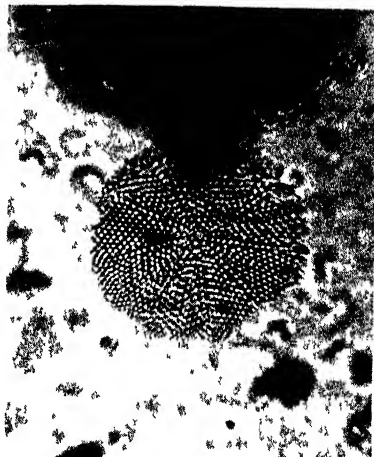


FIG. 113—Photomicrograph of muck soil

Figure 114 shows the effect of the relative rate and ultimate settlement under equal load, and the expansion upon the removal of load, upon the thickness of a soil layer.

The inability of the mechanical analysis to throw light on all the properties is illustrated by Table 27. Here the materials (Table 26) are listed with respect to their fineness and also the extent to which they possess the several physical properties just designated under 1 to 7, inclusive.

In almost every column the arrangement is different, thus indicating that complete identification of the soil can be accomplished only by tests sufficient in number and varied enough in scope to disclose the extent to which all five dominating physical characteristics (see Fig. 115) are present.

Uniform Materials Arranged in Groups.—Based upon the presence of soil constituents, physical properties, and performance, subgrades have been tentatively arranged in eight groups of uniform soil.

The eight groups of uniform subgrade soil are defined as follows:

Group A1.—Well-graded material, coarse and fine, excellent binder. High internal friction, high cohesion, no detrimental shrinkage, expansion, capillarity, or elasticity. Highly stable under wheel loads, irrespective of moisture conditions.

Group A2.—Coarse and fine materials, improper grading or inferior binder. High internal friction and high cohesion only

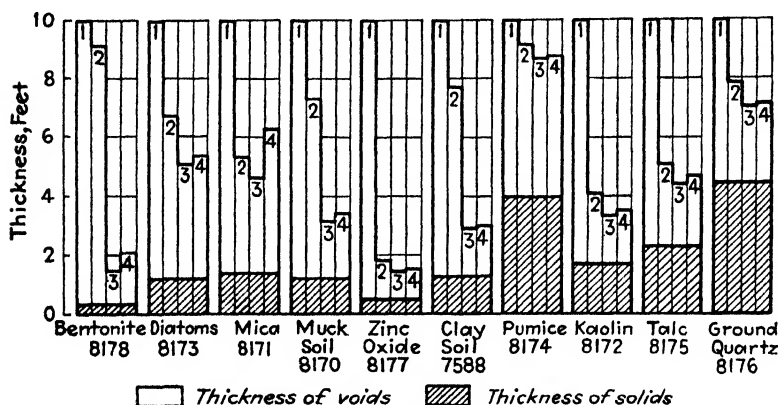


FIG. 114.—Relative thicknesses of layers of different materials when a layer 10 ft. thick at the flocculation limit (1), is compressed by a load of 3 tons per sq. ft. for 30 days (2), to ultimate consolidation (3), and when expanded on removal of load (4).

under certain conditions. There are two types: the “friable” with less plasticity and the “plastic” with more plasticity than the group A1 materials. To remain stable a friable variety requires the cohesion furnished by capillary pressure and therefore is likely to be highly stable when moist but become loose and dusty in long-continued dry weather. The plastic variety, in contrast, remains stable when fairly dry but is likely to soften owing to rains or capillary moisture prevented from evaporating by an impervious cover.

Group A3.—Coarse material only, no binder. High internal friction, no cohesion, no detrimental capillarity or elasticity. Does not heave because of frost or shrink or expand in appreciable

amount. Lacks stability under wheel loads but furnishes excellent support when loads are distributed.

Group A4.—Silt soil without coarse material and with no appreciable amount of sticky, colloidal clay. Internal friction variable, no appreciable cohesion to moderate amounts, no elasticity, capillarity important. Cohesionless varieties absorb water very readily even when not manipulated. When dry or damp, presents a firm riding surface. Conspicuous for frost heaving during cold weather followed by loss of stability during thaws.

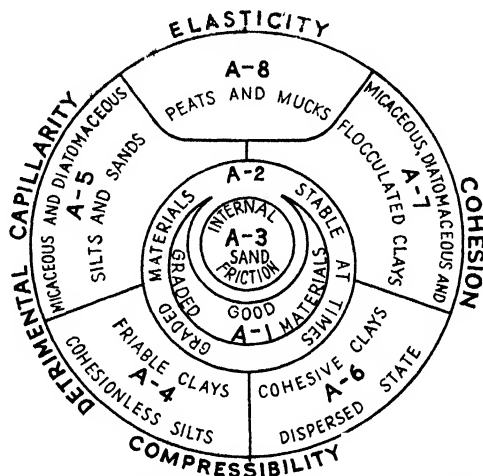


FIG. 115.—Diagram illustrating characteristics of uniform subgrade soil groups.

Group A5.—Similar to group A4 but with highly elastic properties, even when dry. Conspicuous for quick deformations under load and rebound on removal of load. This interferes with proper compaction of macadams and similar types during construction and with retention of good bond afterward.

Group A6.—Clay soils with low internal friction, cohesion high under low moisture content, no elasticity, high expansion and shrinkage properties. In stiff or soft plastic state absorb additional water only if manipulated. May then work up into the interstices of macadams or cause failure due to sliding in high fills. Furnish firm support essential in properly compacting macadams only at stiff consistency. Deformations occur slowly, and removal of load causes very little rebound. Alternate

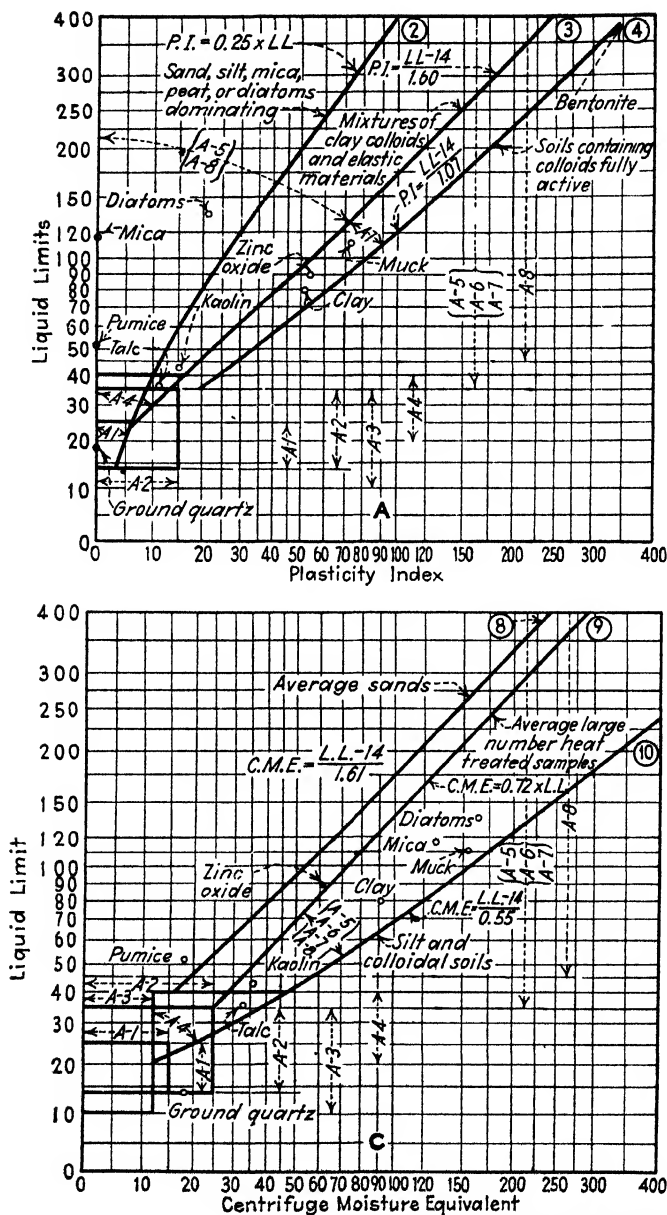
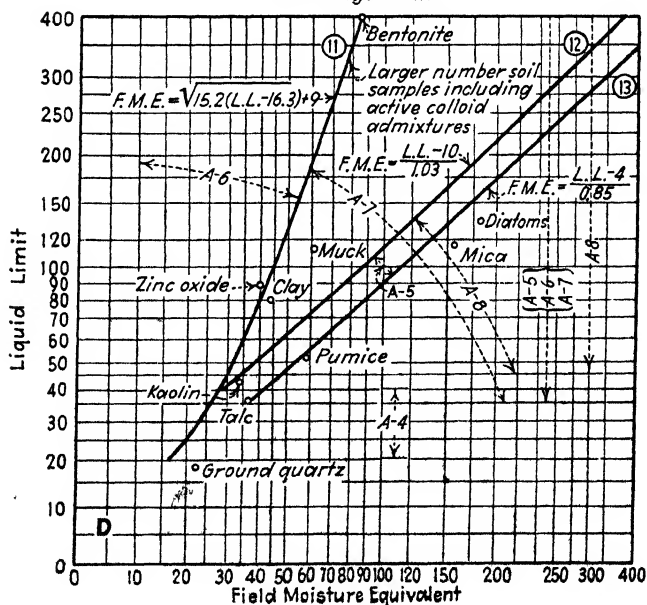
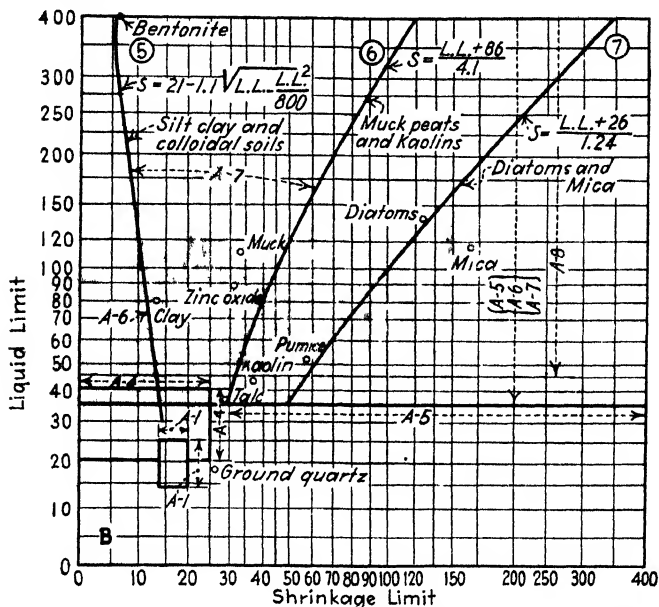


FIG. 116.—The subgrade soil



identification chart.

wetting and drying under field conditions likely to cause considerable volume change.

Group A7.—Similar to group A6 but is elastic at certain moisture contents, deforming and rebounding quickly upon application and removal of load. More detrimental volume changes than in group A6 subgrades. Causes concrete pavements to crack and fault. May contain lime or associated chemicals productive of flocculation in soils.

Group A8.—Very soft peat and muck. Low internal friction, low cohesion, with detrimental capillarity and elasticity. Incapable of supporting a road surface without settlement.

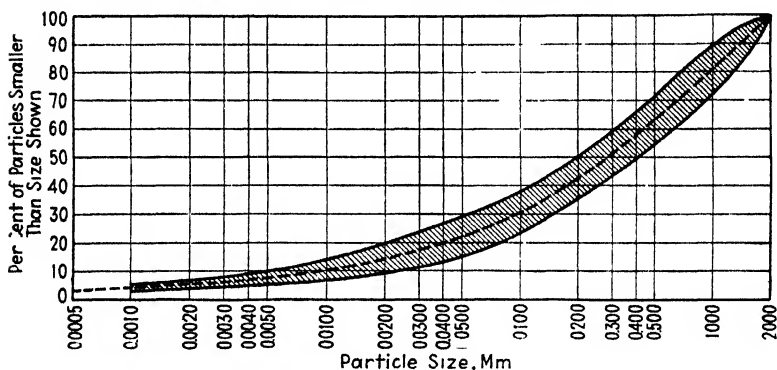


FIG. 117.—Grading of good soil mortars.

Figure 115 illustrates these groups diagrammatically. Different positions on the diagram represent different combinations of the five basic physical soil properties. The center of the diagram represents the perfect A3 subgrade, having a maximum amount of internal friction. As the outer boundary of the diagram is approached the magnitude of internal friction decreases gradually from a maximum to negligible amounts. Moving along the circumference from the bottom to the top of the chart, the indicated soil changes gradually from the compressible to the elastic type.

Adding compressible materials in increasing amounts gradually changes a group A3 sand, first, to a nonplastic variety of the A2 subgrade; second, to a well-graded A1 subgrade; third, to a plastic variety of the A2 subgrade; and, finally, to either a group A4 or a group A6 subgrade. In a similar manner, additions of elastic materials to silts produce A5 soils; to clays, A7 soils; and in large amounts to both silts and clays, A8 soils.

Chart Assists in the Identification of Disturbed Soils.—The four graphs of Fig. 116 constitute the soil-identification chart. They show relations that exist between the liquid limit and four other test constants: the plasticity index, the shrinkage limit, the centrifuge moisture equivalent, and the field moisture equivalent. The gradings and the constants of the eight groups are reviewed as follows:

Group A1.—Grading: material retained on the No. 10 sieve not more than about 50 per cent. The soil mortar—that fraction passing the No. 10 sieve—to consist of clay, 5 to 10 per cent; silt, 10 to 20 per cent; total sand, 70 to 85 per cent; coarse sand, 35 to 50 per cent. Average effective size approximately 0.01 mm., and uniformity coefficient greater than 15. The band (Fig. 117) shows the grading of good soil mortars graphically.

Constants: liquid limit not less than 14 or greater than 25; plasticity index approximately equal to that indicated by curve 2 (Fig. 116A) and seldom larger than 8; shrinkage limit seldom smaller than 14 or larger than 20; and centrifuge moisture equivalent not apt to be larger than 15.

Group A2.—Grading: not less than about 55 per cent of sand in the soil mortar.

Constants: liquid limit not greater than 35; a plasticity index of zero, with a significant shrinkage limit or a plasticity index greater than zero and less than 15 with or without a significant shrinkage limit; centrifuge moisture equivalent not larger than 25.

Group A3.—Grading: effective size not likely to be less than 0.10 mm.

Constants: liquid limit not likely to exceed 35 in appreciable amount; no plasticity index; no significant shrinkage limit; centrifuge moisture equivalent smaller than 12.

Ability of sands to resist sliding when wet indicated as follows:

Liquid limits 10 to 14 signify beach and other rounded sands which slide easily; 30 to 35 indicate rough angular particles not apt to slide easily. In addition, liquid limits when lower than field moisture equivalents indicate those materials that flow because of partial saturation; when equal to the field moisture equivalents, the liquid limits indicate average sands which flow because of full hydrostatic uplift; and liquid limits when greater than field moisture equivalents indicate rough-grained sands

which flow only when in a state less consolidated than that represented by the field moisture equivalent (see Fig. 118).

Group A4.—Grading: less than 55 per cent sand.

Constants: liquid limit not likely to be smaller than 20 or larger than 40; plasticity index not larger than those indicated by curve 3 (Fig. 116A); shrinkage limit not likely to be larger than 25; centrifuge moisture equivalent likely to approach those indicated by curve 10 (Fig. 116C), between 12 and 50; when greater than liquid limit indicates varieties of soils likely to be especially unstable in the presence of water; field moisture equivalent equal to or somewhat larger than those indicated by curve 11 (Fig. 116D) with a maximum of about 30.

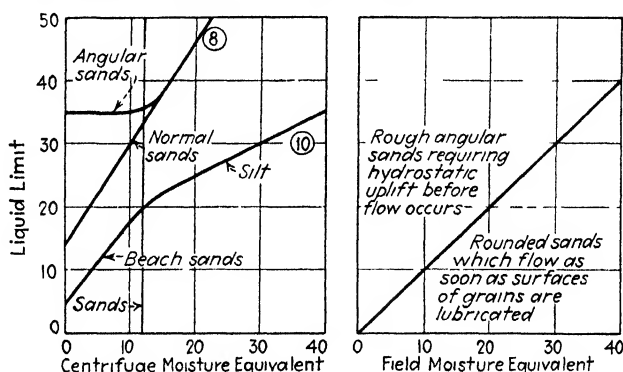


FIG. 118.—Significant relations between test constants for group A3 subgrade soils.

Increase in expansive properties generally indicated when shrinkage limits exceed 20 and approach those represented by curve 6, especially likely to be true when field moisture equivalent exceeds centrifuge moisture equivalent.

Group A5.—Grading: less than 55 per cent sand. (Exceptions.)

Constants: liquid limit usually larger than 35; plasticity index seldom larger than those indicated by curve 3 (Fig. 116A); centrifuge moisture equivalent greater than 12—often between curves 9 and 10—and not likely to waterlog. (Exceptions occur.)

Shrinkage limit likely to be greater than 30 generally and greater than 50 for very undesirable members of this group. In addition, likely to approach those indicated by curve 6 for silts containing peat and to approach those indicated by curve 7 for soils containing either diatoms or mica in appreciable amount.

Field moisture equivalent likely to approach those indicated by curve 12 for silts containing peat in appreciable amount and those indicated by curve 13 for highly elastic soils containing mica or diatoms in appreciable amount.

Group A6.—Grading: seldom contains less than 30 per cent clay.

Constants: liquid limit usually larger than 35; plasticity index approximately represented by curve 4 (Fig. 116A); shrinkage limit not apt to be appreciably larger than that indicated by curve 5 (Fig. 116B); centrifuge moisture equivalent generally waterlogged and likely to equal those between curves 9 and 10 (Fig. 116C); field moisture equivalent not likely to exceed those indicated by curve 11 (Fig. 116D) but may be appreciably smaller for certain colloidal soils. In addition, the volumetric change is not likely to be smaller than 17.

Group A7.—Grading: seldom contains less than 30 per cent clay.

Constants: liquid limit usually larger than 35; plasticity index varies between those indicated by curves 3 and 4 (Fig. 116A); shrinkage limit likely to vary between those indicated by curves 5 and 6 (Fig. 116B); centrifuge moisture equivalent varies between those indicated by curves 9 and 10 (Fig. 116C); waterlogging in centrifuge test may not occur even at very high moisture equivalents. Field moisture equivalent larger than those indicated by curve 11 (Fig. 116D).

Relatively low shrinkage limits with high field moisture equivalents indicate presence of colloidal organic matter, and relatively high shrinkage limits indicate the possibility of frost heave.

Group A8.—Grading: not significant.

Constants: liquid limit larger than 45; plasticity index likely to be smaller than those indicated by curve 3 (Fig. 116A); shrinkage limit indicated approximately by curve 6 (Fig. 116B); centrifuge moisture equivalent between curves 9 and 10 (Fig. 116C); field moisture equivalent apt to be greater than those indicated by curve 12 (Fig. 116D).

Waterlogging in the centrifuge test indicates the mucks containing clay and colloids, whereas very high equivalents without waterlogging indicate peat not more than slightly decomposed.

Group Numbers Indicate the Performance of Soil as Foundation for Footings, Abutments, and Embankments.—The manner in which use of the foregoing classification may assist in practice

may be illustrated by a brief review of the requirements of earth used as foundation or in various types of structures.

The problems differ depending on whether the foundations are for piers or footings, abutments, or embankments. With piers or footings we are concerned primarily with settlements, which depend upon the compressibility of the soil. The allowable settlement for the particular structure having been decided upon, and the compressible characteristics of the undersoil determined, it remains only to compute the required size of footing. If the footing of the required size cannot be used owing to practical considerations, several possibilities are open whereby the size of footings might be reduced. Piling or caissons might be used to the depth of rock or firm undersoil or in such soils as are not reduced in bearing value by the driving of piles; piling not extending to rock might be of benefit by increasing the size of the pressure bulb and thus reducing the stress. The third possibility is the use of some sort of injection method of stabilization in which piling may or may not be used. The best grade foundation soils belong to the A1 to A4 groups inclusive. The less plastic varieties of the A6 and A7 groups are next best, and soils of the A5, A8, and the more plastic varieties of the A6 and A7 groups are the soils that as a rule require special treatment.

In addition to settlement vertically, lateral flow of the soil may prove troublesome to abutments, because such structures are required to perform the functions of retaining walls as well as piers. Consequently in the design of such structures for use on the softer soils, such as those of the A8 group and the more plastic varieties of the A6 and A7 groups, provision should be made to prevent the abutment's being displaced laterally when surcharge back of the wall causes flow of the soft undersoil toward the stream channel.

Use of Classification in Selection of Fill and Dam Materials. Generally, soils of the A1, A2, A3, and the better varieties of the A4 group can be expected to perform satisfactorily in fills without regard to moisture content and special methods of consolidation. Consequently, they are suitable alike for use in either rolling or hydraulic methods of construction.

Certain soils of other groups such as the moderately plastic varieties of the A4, A6, and A7 require the use of the densification method of stabilization, in construction, to insure satisfactory

performance. The extra cost of the special methods of construction might warrant a longer haul in order to obtain the better grade materials referred to above. If it is possible to avoid doing so, certain soils such as those of the A5 and A8 groups, the more plastic varieties of the A6 and A7 groups, and the least stable varieties of the A4 group should not be used in fill construction. However, if used, they must be densified according to rigid specifications.

In earth-dam construction, there should be a low degree of permeability and a high degree of stability. A homogeneous embankment of low percolation rate may be used, or there may be an impervious core wall with more porous material on each side. Groups A1, A2 plastic and A4 plastic and the better varieties of the A6 and A7 groups are suitable for either the entire dam or only the core wall. Friable A2 and A4 soils are suitable for use in the porous portions. Groups A3, A5, A8, and the highly plastic varieties of the A6 and the A7 soils should not be used in dam construction. The densification method of construction is desirable whenever soils of considerable plasticity are used; otherwise, compaction by rolling or placement by the hydraulic method is suitable.

Role of Soil Grouping in the Field of Highway and Airport Construction.—It has been suggested that each and every particular pavement variable—stability, “beam” strength, pavement thickness, steel reinforcement, grooves and joints, shoulders, subbases, subgrade treatments, subgrade preparation, and artificial drainage—has a particular function to perform with regard to the conditions of support furnished by the subgrade.

Rolling the Subgrade.—The character and condition of the subgrade soil determines the extent of rolling required. A small amount of rolling to smooth off sands after they have been thoroughly sprinkled seems to be sufficient. Compacting silts and fine-grained clays with a very heavy roller breaks down their natural structure and thus, to the depth affected by rolling, increases their support.

Micaceous and diatomaceous silts, coarse-grained gumbo, and fine-grained clays containing organic matter rebound upon removal of load. Consequently, these subgrades are not permanently compacted by rolling. The micaceous friable soils may be compacted slightly when wet but are likely to expand upon

drying. Owing to their low density these subgrades furnish low support when either wet or dry. The adobes, in contrast, are hard and compact when dry and may be temporarily compacted when damp. They expand very appreciably, however, when subjected to water, such as may be furnished, for instance, by capillarity or even that furnished by newly laid concrete.

Thus, in order to prevent detrimental volume change, an attempt should be made to obtain uniform density in elastic subgrades and to prevent their absorbing moisture during the setting of the concrete.

Porous Base Courses and Drainage.—According to the Highway Research Board (129), base courses of cohesionless gravel, field stone, etc., laid on clays not only fail to benefit rigid pavements in all cases but have been known to increase the extent of heave due to frost and the extent of breakage. This can easily be explained. Base courses containing large voids permit the evaporation and thus promote the flow of capillary moisture in the subgrade. Also, they provide space in which free water entering from the road surface or from the subgrade and that furnished by the condensation of capillary moisture may accumulate. Thus in the absence of drainage outlets, or when ice or lack of proper maintenance prevents drainage outlets from functioning, porous base courses serve as reservoirs under the road surface. Under these conditions more water to produce frost heave is furnished by the presence than by the absence of porous base courses. Furthermore, the large voids permit the penetration of the soft clay from beneath during thaws and may cause the base course to settle unequally, thus furnishing non-uniform support productive of pavement failure.

To lead out water furnished by springs through separate drains and to intercept that furnished by water-bearing strata with trenches and tile drains placed under the edges of the pavement seems preferable to the indiscriminate use of porous bases for drainage purposes. Except where rock subgrades furnish water through a number of crevices and where the thickness of base course can be made approximately equal to the depth of frost penetration, base courses containing large voids may not prove beneficial.

Compacted Base Courses.—Layers of well-graded sand clay, gravels with binder, and compacted slag and crushed stone as

undercourses for rigid pavements may prove beneficial and as foundations for nonrigid pavements are, of course, a necessity.

Tightly compacted base courses—layers of sand clay, for instance—prevent the entrance of water into the subgrade from above and also the escape of capillary moisture by evaporation. They should, therefore, be more effective than porous base courses for preventing detrimental volume changes in subgrades having high shrinkage properties. Separating the sand-clay layer from the subgrade by a coating of light bituminous material will probably furnish additional protection against volume change in the subgrade.

When the subgrade support is low but uniform, increasing the strength of concrete pavements or bases seems preferable to the use of subbases for obtaining insurance against load cracking.

Requirements for soils of the different groups may be summarized as follows:

Group A1 Soils.—Drainage to prevent frost heave when ground-water elevation is high. Suitable for stabilized soil road when treated with deliquescent chemicals. Wearing course sufficient as pavement.

Group A2. Friable.—Drainage not required. Stabilized road material for use under moist conditions. Serves excellently as base course for bituminous surface treatment. Thin wearing courses in pavement.

Group A2. Plastic.—Drainage to prevent frost heave and softening of binder from below. Treatment with deliquescent chemicals or suitable as base for thin wearing courses only under dry or arid conditions.

Group A3. Coarse Materials.—Subgrade treatment by admixture of binder or light tars and substantial wearing course. Otherwise, moderately thick nonrigid, or thin rigid courses. Drainage not required.

Group A4. Silts.—When naturally drained or when artificial drainage is possible: thick macadam or concrete pavement of medium thickness (not less than 8-6-8). Subgrade treatment by admixture of coarse constituents permits reducing thickness of macadam. Oiling combined with subgrade treatment may further improve the quality.

When there is a high ground-water level, and drainage is not possible: macadam unsuitable. Thick concrete pavement (not

less than 9-7-9), crack control, and reinforcement. Oiling not promising because water comes from below. Subbase may be beneficial for reducing frost effect.

Group A5.—Same as for group A4, wet. Condition still more unfavorable.

Group A6.—Distinguished by the state of the soil, whether impermeable (homogeneous) or permeable (full of cracks and root holes).

Homogeneous State.—Ample load distribution by thick macadam or rigid pavement. Degree of required load distribution depends on degree of softness. Surface treatment (oiling or screenings or both) prevents material from working into non-rigid base course. Crack control for reducing effect of unequal shrinkage.

Permeable State, Drainage Feasible.—Macadam or rigid type. Subgrade treatment by mechanical manipulation under traffic increases stability.

Permeable State, Drainage Not Feasible.—Very strong macadam or rigid type with crack control. Reinforcement desirable. Subgrade treatment by admixture of coarse material on subbase for reducing frost heave.

In Fills.—Mechanical manipulation by traffic very beneficial. Also subgrade treatment by admixture of coarse constituents. Place the fill in dry season. Springs entering the base from below should be piped away. Treatment by oiling may reduce danger of saturation from above.

Group A7.—Surface treatment by mechanical manipulation for preventing unequal expansion and by application of tar paper for preventing expansion beneath fresh concrete. Otherwise, treat them like the soft, homogeneous group A6 subgrades.

Group A8.—Fill on top of soft ground, according to Michigan and Minnesota practice (see pages 324 to 329). Pavement requires "beam" strength, ample crack control, and reinforcement.

Problems

1. What is the significance of the routine test results?
2. What properties of soil constituents are disclosed by the liquid limit, the plasticity index, and the field moisture equivalent?
3. Define the eight groups of uniform soil.
4. To what groups belong soils with the gradings and test constants listed below?

Test data	Sample number				
	1	2	3	4	5
Grading of fraction passing No. 10 sieve:					
Coarse sand, per cent.....	3	0	0	38	0
Fine sand, per cent.....	5	2	18	40	6
Silt, per cent.....	32	60	46	14	28
Clay, per cent.....	60	38	36	8	66
Constants of fraction passing No. 40 sieve:					
Liquid limit.....	56	34	72	20	68
Plasticity index.....	30	10	27	3	46
Shrinkage limit.....	24	20	54	19	13
Shrinkage ratio.....	1.5	1.8	1.1	1.7	1.9
Centrifuge moisture equivalent.....	47	31	68	12	86 ¹
Field moisture equivalent.....	37	27	65	14	32

¹ Waterlogged, free water included.

5. Describe how the soil grouping may be of assistance in design of foundations.

6. Under what conditions should subgrades be compacted by the use of heavy rollers?

7. When should compacted base courses be used?

CHAPTER XII

DESIGN OF GRADED MIXTURES

Studies in soil stabilization are concerned with the effect of treatments for influencing (a) the ratio of free to adhesive moisture in the soil mass, (b) the degree of adhesion of the binder films to the soil particles, and (c) the permanency of the joint developed by the binder film and soil particle. The methods include compaction, densification, use of admixtures, electrochemical treatment, and application of heat.

Compaction and Densification.—Methods used in the consolidation of stabilized earthworks may be of two kinds depending upon such factors as the character of the soil materials used, the type of admixtures employed, if any, and the purposes that the structures are to serve. They are termed “compaction” and “densification.”

Granular material and clay soils are affected differently by traffic. The wagon or truck wheels will sink into roads of either pure sand or soft, sticky clay. However, granular particles have a tendency to compact, whereas clays never lose the tendency to displace under traffic and form ruts. Because of this, a load applied at the top of an unconsolidated or partially consolidated layer of a graded mixture which contains enough granular material to provide for mechanical interlocking is readily transmitted vertically and causes consolidation throughout the thickness of the layer. Consequently, any means of supplying adequate load, whether it be a flat-wheeled roller, a crawler-type tractor, truck graders, trucks, or traffic, will assist in the consolidation of graded mixtures.

The term compaction is used to designate this type of consolidation. The soil is rolled in relatively thin layers and as a rule in moistened state, but neither the thickness of layers nor the moisture content is critical. The density produced by this method may be exceedingly high but is nevertheless of qualitative character. It may be considerably increased by the action of traffic after construction. When the coarser particles

are absent or are present in such amounts that they are surrounded by the cohesive fine-grained particles, the effect of a load applied to a layer of such material is to consolidate that portion immediately beneath the load without adequately compacting the particles in the lower part of the layer. Consequently, rolling fine-grained soils with a flat-wheel roller may produce undesirable stratification and insufficient consolidation. It requires the prongs of a sheep's foot or tamping-type roller, or equipment that produces similar effect, to penetrate the unconsolidated material and compact the layer from the bottom upward in order for the material to be consolidated uniformly throughout its thickness without stratification.

Densification is used to designate the consolidation produced by the use of this type of rolling equipment passing a given number of times over soil deposited in layers of specified thickness and at a particular moisture content. The moisture content is critical, and the corresponding density is quantitative in character. Both are predetermined by test. The required density is produced as part of the construction procedure. A considerable increase in the density of the structure after construction is not to be anticipated.

Densification Tests.—The effect of consolidation on the density of soil may be determined by the tests devised by R. R. Proctor (38). They are based on the fact that for each soil there is but one moisture content, termed the "optimum" moisture content, at which the maximum density is produced by a specific degree of compacting. For every density, each soil has a particular stability, as indicated by the force required to penetrate the soil with a footing of known area at a given rate of speed. When the soil is compacted to maximum density at optimum moisture content, the adsorptive attraction between water and soil particles is probably completely satisfied so that the tendency for moisture to enter the soil and expand or soften the soil mass is largely eliminated.

The apparatus required for the compaction test consists of a brass cylinder 4 in. in diameter and about $4\frac{1}{2}$ in. deep, which is mounted on a removable base plate and fitted with a detachable collar 2 in. high to hold the loose soil in place while compacting; a $5\frac{1}{2}$ -lb. cylindrical rammer with an end area of about 3 sq. in.; and a plasticity needle (see Fig. 119).

In the compaction test approximately 5 lb. of dry soil passing the No. 10 sieve is mixed thoroughly with just enough water to make it slightly damp and compacted in the cylinder in three layers, each layer receiving 25 blows from the rammer dropped from a height of 1 ft. The soil is then struck off to the level of the cylinder, weighed, and the stability determined with the plasticity needle by measuring the pressure required to force it into the soil at the rate of $1\frac{1}{2}$ in. per second. A small sample of the compacted soil is oven dried to determine the moisture content.



FIG. 119.—The Proctor compaction-test apparatus.

This procedure is repeated, each time adding about 1 per cent more water, until the soil becomes very wet and there is a sub-

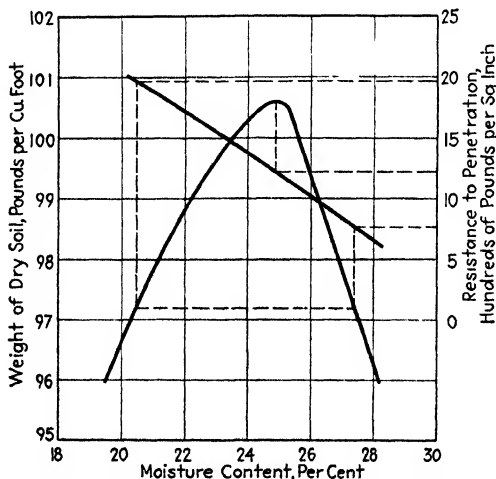


FIG. 120.—Data furnished by Proctor compaction test.

stantial decrease in the weight of the compacted soil. The effect of moisture on the densities of the compacted samples is shown by

plotting the densities of the compacted soil, expressed in pounds per cubic foot, against moisture content. The plasticity needle readings, expressed in pounds per square inch, are also plotted against moisture content to show the effect of moisture on stability.

Two curves resulting from the test (Fig. 120) illustrate the significance of the compaction data. The dry-weight, moisture-content curve discloses that for this soil a moisture content of about 25 per cent is required, if maximum compaction is to be obtained. The corresponding stability is about 1,200 lb. per square inch.

If, at the specified compaction, the stability of this particular soil is indicated by the plasticity needle to be higher than 1,200 lb., the increase can be considered as only temporary if the fill is to be unprotected from water after construction. Thus, a stability of 1,950 lb. per square inch indicates a moisture content of slightly more than 20 per cent. This corresponds to a dry weight of about 97 lb. per cubic foot. At this density the soil can take up moisture to a maximum of slightly more than 27 per cent which, in turn, corresponds to a stability of but 750 lb. per square inch.

Penetrometer for Testing Stabilized Soils.—The Proctor tests have proved suitable for use in the field for purposes of control during the construction of earth fills. For laboratory purposes, however, the plasticity needle, as originally designed, is of limited value because of the possibility of prohibitive personal error. This is due to several causes: (a) It is almost impossible to obtain truly vertical movement during penetration; (b) the position in which the operator is forced to stand makes accurate measurement of the depth of penetration very difficult, especially when the soil bulges adjacent to the area penetrated; (c) a constant rate of penetration is very difficult to attain, partly because of item (b) but more because the movement of the handle, owing to deflection of the force-indicating spring, can be several times the movement of the needle footing. In addition, the resistance to penetration, even at the optimum moisture content, is often found to be greater than the maximum of 2,000 lb. per square inch which can be measured with the original needle.

In the improved apparatus developed at George Washington University for obtaining information on the stability, capillarity, and permeability of compacted soils, provision has been made

for supplementary attachments for obtaining information on the swell and compression characteristics of compacted soils.

The design shown in Fig. 121 is essentially that of a press which insures vertical movement of the plunger, accurate measurement of the depth, and, hence, control of the rate of penetration with provision for applying constant load. The pressures are determined by use of a platform balance of 300 lb. capacity and not

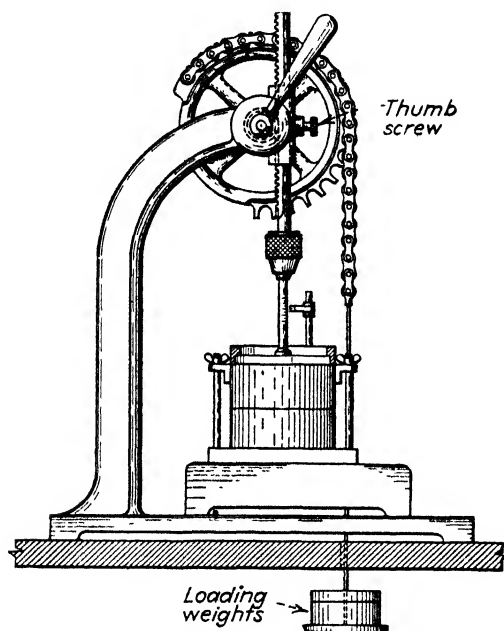


FIG. 121.—Penetrometer for testing stabilized soils.

subject to appreciable vertical movement. Depth of penetration is measured by a pointer referring to graduations on the needle stem. There are seven penetration footings with end areas ranging from $\frac{1}{20}$ to 1 sq. in. Pressures up to 6,000 lb. per square inch can be measured.

The supplementary parts (shown in Fig. 122) include (a) one compaction cylinder of 4 in. inside diameter and $6\frac{1}{2}$ in. high made in three segments, the bottom one 2 in. high, the middle one $2\frac{1}{2}$ in. high, and the top one 2 in. high, as shown in Fig. 122a; (b) a base plate $\frac{3}{4}$ in. thick and 6 in. square, as also shown in Fig. 122a; (c) one filter base plate 6 in. square and 1 in. thick machined

so as to receive a filter stone 4 in. in diameter and $\frac{1}{2}$ in. thick and to permit the free flow of water to and from the bottom of the filter stone, as shown in Fig. 122c. Attached to this filter base is a combination permeameter and capillometer which consists of a graduated glass standpipe $\frac{1}{2}$ in. in diameter, a vacuum gage, one three-way valve, and one needle valve connected by tubing as shown. Also, as shown in Fig. 122c, the upper segment of the

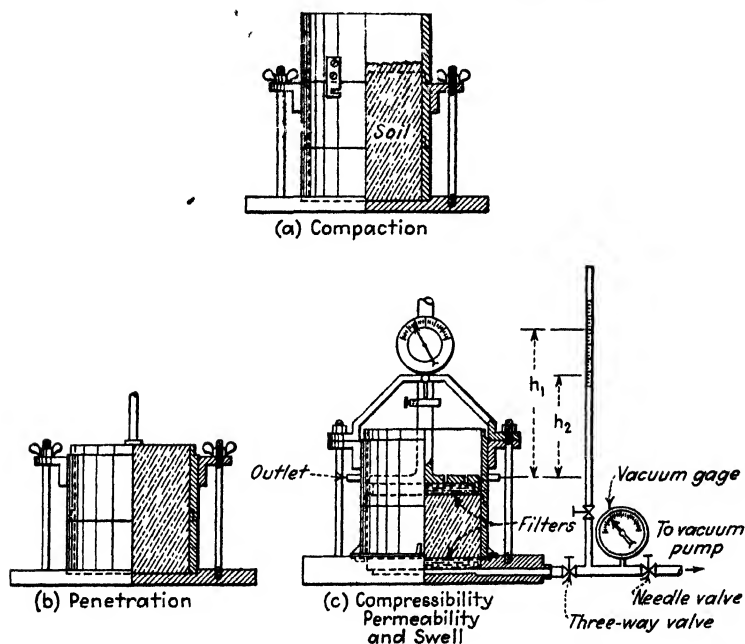


FIG. 122.—Supplementary attachments to penetrometer shown in Fig. 121.

cylinder, required when the apparatus is used as a permeameter is provided with an outlet pipe. A further attachment is a plunger fitted with porous filter stone, as shown in Fig. 122c, and machined to fit neatly as shown.

Essentials of Test Procedure.—The proposed complete testing procedure consists of the following steps: (a) With the apparatus arranged as shown in Fig. 122a, the sample is compacted in the manner previously described for the compaction test. (b) The sample is struck off level with the top of the second segment, as shown in Fig. 122b. (c) The weight of the compacted sample is

determined. (d) The plunger is forced into the sample, readings of the balance being taken at intervals of $\frac{1}{4}$ in. penetration to a maximum of 2 in. (e) The upper segment is detached, the sample struck off level with the top of the lower segment, and a small portion of the material removed in leveling off the sample is used to determine the moisture content. (f) The solid metal base plate is replaced by the filter base plate, the segment with outlet pipe is attached, the filter plunger fitted into position, the thumbscrew tightened, and the permeability and capillarity attachments connected as in Fig. 122c. The three-way valve at the bottom of the standpipe is opened, and the standpipe kept filled until water escapes from the outlet pipe. To determine the permeability, the time required for the water level to drop a given distance in the standpipe is then observed. (g) The force produced by the tendency of the soil to swell may be read on the scale. The thumbscrew holding the plunger in position is now loosened, and the amount of swell of the sample is observed. (h) The compressive and expansive properties of the sample may be determined by observing the deformation-load and the deformation-time relationships under conditions of both increasing and decreasing load applications. (i) At the conclusion of the loading test, if made, or otherwise at the conclusion of the permeability test, the maximum capillary rise is determined in a manner based upon a method originally suggested by J. H. Englehardt (130), a modification of which has been described by Hogentogler and Willis (70).

The three-way valve is adjusted so as to close off the standpipe and provide access of water into the soil by connecting to a reservoir. By means of the needle valve, the vacuum is increased by increments until abrupt loss of the vacuum thus built up indicates that the moisture films in the sample have been broken.

Calculations are made, and the results recorded as follows: The density of the wet sample in pounds per cubic foot is determined by dividing its weight in pounds by its volume ($\frac{1}{30}$ cu. ft.). The corresponding dry density of the sample in pounds per cubic foot is determined by the formula

$$\text{Dry density} = \frac{\text{wet density}}{100 + \text{moisture content}} \times 100 \quad (71)$$

The dry weight-moisture content and stability-moisture content relations are recorded graphically, as in the Proctor tests.

The permeability is determined on the basis of the relations expressed in formula (30) (Chap. VI).

The maximum capillary height in feet is given by multiplying the final reading of the vacuum gage (in pounds per square inch) by 2.3.

Densified Soils Undergo Four Stages of Wetting.—The work at George Washington University shows that when the moisture contents are expressed as percentages of the combined volume of

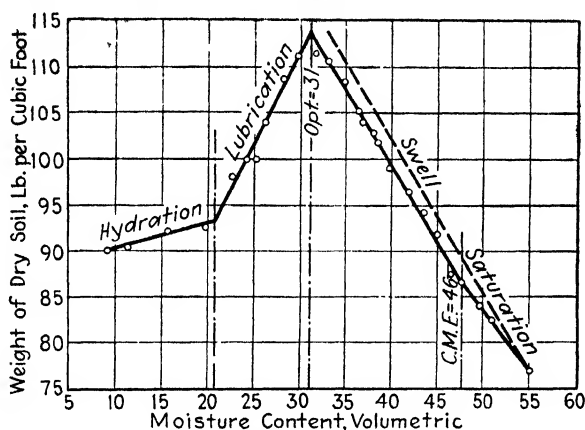


FIG. 123A.—Relation of density to moisture content in per cent by volume (soil No. 1).

soil solids and moisture, instead of the weights of soil solids as in Fig. 120, the density-moisture content relation becomes a series of straight lines with different slopes as shown by the full-line curve in Fig. 123A.

The relation between the two moisture contents is given by the equation:

$$w' = \frac{w}{w + \frac{100}{G}} \times 100$$

where w' = moisture content in percentage of combined volume of solids and moisture.

w = moisture content in percentage of weight of solids.

G = specific gravity of solids.

The broken-line curve (Fig. 123A) shows what the relation of the density to the moisture content of a soil would be if the samples contained no air. At any density the difference in moisture contents indicated between the two curves represents the percentage of moisture, by volume, required to replace the contained air.

The moisture contents at which the straight lines of the full-line curve intersect indicate the limits of four distinct stages of wetting which the compressed samples of a soil (hereinafter

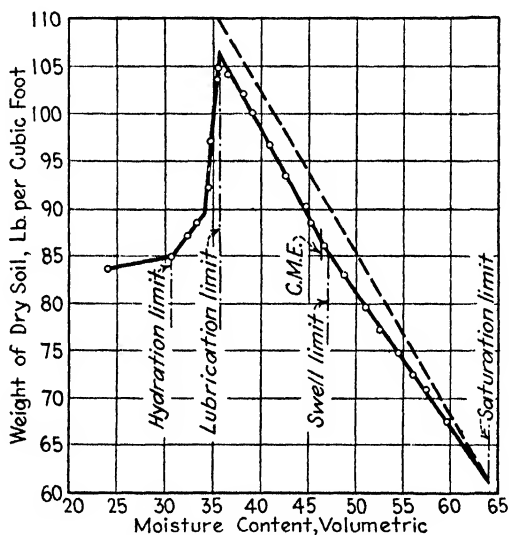


FIG. 123B.—Relation of density to moisture content in per cent by volume (soil No. 2).

referred to as "soil No. 1") undergo before the pores become completely filled with water.

Wetting up to a moisture content of 20.7 per cent, Fig. 123A, may be termed the stage of hydration. During this stage part of the contained water is absorbed by the soil particles and the remainder adsorbed on their surfaces in the form of cohesive films. The maximum moisture content of this stage of wetting is termed the "hydration limit," for convenience.

Moisture contents ranging from 20.7 per cent to 31.0 per cent indicate the stage of lubrication. Part of the contained moisture now acts as a lubricant to facilitate the rearrangement of particles

being compacted into closer association without, however, excluding all the air. The maximum moisture content of this stage of wetting may be termed the "lubrication limit." It is the optimum moisture content at which maximum density is attained.

Water in excess of 31.0 per cent causes the soil mass to swell, although the air contained at the optimum moisture content is not appreciably decreased until the "swell limit," a moisture content of 47.7 per cent in this case, is reached.

Moisture contents between 47.7 per cent and the "saturation limit," 55.0 per cent in this case, represent the stage of saturation. During this stage practically all the air is displaced and the soil becomes truly saturated.

Some soils have an additional stage of wetting between the hydration and the lubrication stages as shown in Fig. 123B ("soil No. 2").

From the foregoing, the stability-moisture content relation would be expected to be a series of straight lines also. Figure 131, discussed in Chap. XIII, shows that this is true if the moisture contents (by volume) are plotted against the logarithms of the penetrometer readings.

Effect of Film Thickness on Densification.—Simple mathematical computations, involving assumptions not strictly true but valid enough for illustrative purposes, can be used to estimate the surface area of the soil particles in a given mass.

The soil (Fig. 123B) consists of sand, 44 per cent, silt, 12 per cent, and clay, 44 per cent. The estimated surface area at the hydration limit, assumed to be 30.6 per cent, is 235,000 sq. ft. per cubic foot of soil mass.

If the water and air were distributed uniformly over the entire surface of the soil particles, the average thickness of water film would equal about 11/1,000,000 in. and of the air film 12/1,000,000 in. A thickness of about 5/1,000,000 in. represents the hygroscopic moisture contained in the air-dried sample.

In like manner the thicknesses of the moisture films at the lubrication, swell, and saturation limits may be estimated to be about 14/1,000,000 in., 22/1,000,000 in., and 46/1,000,000 in., respectively.

Films differing several millionths of an inch in thickness might seem of little import on a sand or gravel particle. On a clay

particle as small as $1/10,000$ in. (0.0025 mm.) in diameter, however, such small differences in film thickness become controlling influences on performance of the soil containing the clay.

The diameter of such a particle is compared with the thicknesses of surrounding film in Fig. 123C. During the stage of hydration, the moisture film, more viscous and glue-like than free water, increases in thickness to a maximum of $11/1,000,000$ in.

During the stage of lubrication, the water film increases in thickness to $14/1,000,000$ in., the excess of $3/1,000,000$ in. above the hydration film acting more like free water to facilitate com-

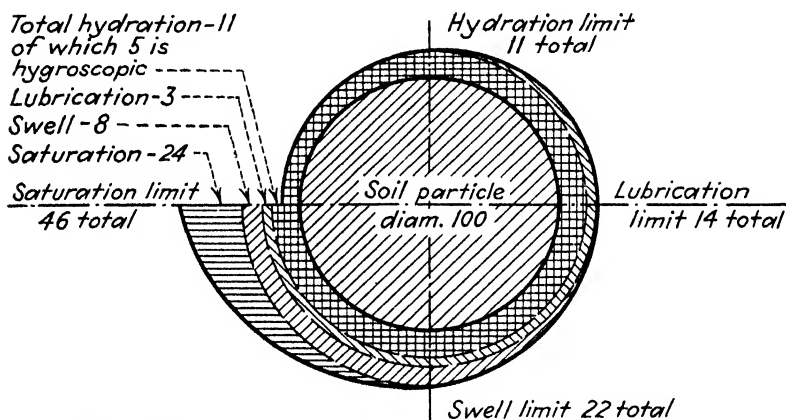


FIG. 123C.—Relative diameter of particle and average thicknesses of moisture films, in millionths of an inch.

paction of the particles. Correspondingly the air content is reduced to about 3 per cent by volume.

During the stage of swell, the water films increase in thickness to $22/1,000,000$ in., the air content of the soil remaining fairly constant. Moisture in films of this thickness and less is possibly attracted more strongly by the soil particle than by gravity. It would then coat the soil particles, as shown in Fig. 123D, without filling the air voids between the outer surfaces of the films.

Moisture in excess of the swell limit possibly is free water attracted more strongly by gravity than to the soil particle surfaces. Consequently, the moisture gradually replaces the contained air until the soil becomes completely saturated at an

equivalent film thickness of $46/1,000,000$ in. in this case. Hydrostatic uplift may not be fully effective until the saturation limit, indicative of saturation, is reached.

In the centrifuge moisture equivalent test the films adhere to the soil particles so strongly that a force of 1,000 times gravity fails to remove them. The estimated thickness of film in soil No. 2, Fig. 123B, at the centrifuge moisture equivalent is about $21/1,000,000$ in.

In computing the average thicknesses just mentioned the effect of size of grain was disregarded. As a matter of fact films on sand grains may be many times as thick as on clay particles.

The average diameter of sand grains passing the No. 10 sieve and retained on the No. 270 sieve is 0.041 in. At a centrifuge moisture equivalent of 3, the estimated film thickness is $560/1,000,000$ in. At 6 per cent moisture, where considerable "bulking" of the sand occurs, the estimated thickness of film is $1,120/1,000,000$ in. This would increase the diameter of the average sand grain about 5.4 per cent. If each dimension of a cube of dried sand were increased by 5.4 per cent, the total volume would be increased 17 per cent, an increase commonly observed due to the bulking of damp sand.

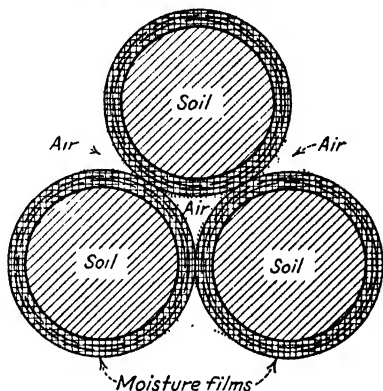


FIG. 123D.—Moisture films and entrapped air.

Admixtures Used in Soil Stabilization.—The kinds of admixtures and the means by which they effect soil stabilization are summarized as follows:

1. Mineral aggregates and soil constituents of the character and size required to make graded mixtures stable.
2. Primes and fillers such as soaps, stone dust, and slag to increase the adhesion between mineral constituents and the chemical and bituminous admixtures and thus assist in retaining the benefits that the admixtures are expected to furnish.
3. Solutions of electrolytes, such as calcium chloride, common salt, sodium hyposulphite, etc., to reduce the thickness of adhesive water films on soil particles and thus provide stabilized mixtures with greater density.

4. Neutralizers such as limestone dust, slag, hydrated lime, etc., which serve to neutralize or "sweeten" acid soils and thus prevent the loss of stabilizing chemicals by detrimental base exchange.

5. Moisture-retentive chemicals, such as calcium chloride and common salt, to provide soil binders with enough moisture to facilitate the compaction of graded mixtures by traffic.

6. Water-insoluble binders, such as Portland cement and bituminous materials, to furnish films more substantial than those of moisture alone and to destroy permanently the colloidal properties responsible for detrimental volume change in soil mixtures.

7. Adhesives such as sulphite liquor, molasses, calcium silicate, and calcium humate.

Stable Soil Mixtures.—The essential features of design and construction of all types of earthworks in which soil mixtures are employed are disclosed by a knowledge of the requirements for stability in road surfaces. In this case the purpose of stabilization is to provide soil roads with enough abrasive resistance and shear strength to accommodate traffic under prevalent weather conditions, without detrimental deformation.

According to Dr. C. M. Strahan (24, 25), the best varieties of graded soil mixtures can be expected to furnish roads that when dry are strong, hard, free from noticeable ruts, holes, or corrugations, possess a well-shaped crown, and drain freely into the side ditches. A thin layer of loose, sandy material may sometimes be present but is not likely to be sufficient in amount to impede traffic. During heavy rains or after long wet spells the surface should not be noticeably softened but should remain practically nonslippery and free from appreciable mud. Driving speed should have to be only slightly reduced.

Admixtures of Mineral Aggregates and Soil Binder.—Proportioning of soil materials for surface courses should provide:

1. Enough crushed rock, gravel, or slag retained on the No. 10 sieve to furnish the strength and hardness needed to resist the abrasive action of traffic.

2. Enough sand or other granular material passing the No. 10 sieve and retained on the No. 200 sieve to provide the interlocking of soil grains and thus increase the shear strength.

3. A quantity of silt and sand (particles 0.074 to 0.005 mm. in diameter) to act as a filler and to provide the capillary bond necessary for stability when the binder clay loses cohesion in wet weather.

4. Enough clay (particles smaller than 0.005 mm. in diameter) to retain the minute adhesive films after the larger films of capillary moisture have

evaporated and thus provide the adhesion required to maintain stability during dry weather.

Physical and Chemical Properties of Aggregate and Binders.

The aggregate of satisfactory stabilized mixtures should be hard and durable enough to resist weathering, traffic abrasion, and crushing. Sound, tough particles or fragments of gravel, stone, slag, or combinations of them, crushed to the proper size, should prove suitable. Certain types of shales and similar materials which break up and weather rapidly when alternately frozen and thawed, or wetted and dried, should not be used.

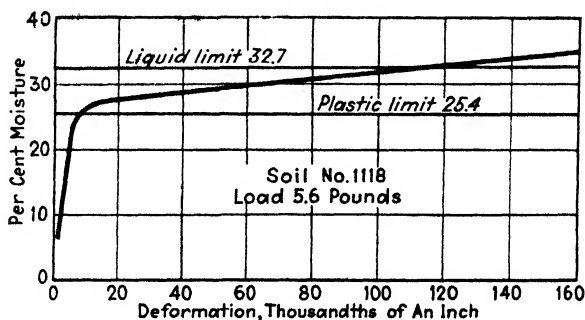


Fig. 123E.—Relation between moisture content and deformation under constant load.

The soil fines should be of a character such as to provide graded mixtures with the proper balance of capillarity and cohesion without risk of detrimental volume change. Fines that swell enough in the presence of moisture to have moisture contents appreciably in excess of their plastic limits are likely to prove the least desirable. This is because the more the moisture content increases above the plastic limit the more likely clay is to become a lubricant instead of a binder.

Just as the adhesive instead of the free water becomes the determining influence when the moisture content of soils is reduced from above to below the plastic limit, there is a corresponding opposite change when the moisture content of clays is raised from below to above the plastic limit. With this change the clay can be considered as changing from a binding to a lubricating medium, as illustrated in Fig. 123E. Here it will be noted that for a constant load, the deformation increases very slowly (about 0.00026 in.) for each 1 per cent in moisture content, until a mois-

ture content approximately equal to the plastic limit is reached. The lubricating effect of the moisture above this amount is indicated by the fact that increase in deformation is thereafter about one hundred times as great (0.022 in.) per 1 per cent increase in moisture content (27).

Finally, the surface characteristics of the entire mixture should be such as to provide satisfactory adherence of chemical or bituminous admixtures to the mineral particles and to prevent the occurrence of detrimental base exchange.

In addition to actual swell test data, such qualities as high plasticity, considerable shrinkage on drying, high moisture equivalents, and the like also indicate soils likely to undergo detrimental volume change.

Neutralizers.—It has previously been explained in Chap. IV that the more acid the soil the greater is its base-exchange capacity and consequently its corrosive properties. Therefore, the best way to insure that detrimental exchange will not occur is to have the soil mixture nearly neutral.

Limestone dust and granulated slag seem admirably suited for use as pretreatments or with other admixtures to neutralize or "sweeten" acid soils. This may be especially desired in the case of siliceous clays. With highly acid conditions (pH as low as 4.0) several pounds of stone dust or slag per square yard of road surface or base will probably suffice to prevent detrimental base exchange. A smaller amount of hydrated lime might also produce the desired result.

Winterkorn's work (78) on exchange capacities (see Tables 7 and 8, Chap. IV) furnishes some idea of the amount of neutralizer that may be required through the determination of the exchange capacities for the various types of clays completely ionized with hydrogen. If complete replacement occurred, his data show the grams of replaceable hydrogen per 1,000 grams of colloid to be as follows: bentonite, 0.85; Putnam clay, 0.65; Wabash clay, 0.78; Iredell clay, 0.35. It is hardly likely that any properly graded road surface would ever contain more than 5 per cent of clay colloid as active as the Putnam. If this is true, then in a road surface 4 in. thick compacted to an initial density of 130 lb. per cubic foot, there would be but 19.5 lb. of colloid per square yard of road surface. In such case there would be $0.00065 \times 19.5 = 0.0127$ lb. of replaceable hydrogen.

The amount of neutralizer required to replace completely the hydrogen equals the molecular weight of the neutralizer divided by the valence of its metal times the amount of replaceable hydrogen.

The molecular weight of calcium carbonate, CaCO_3 , is 100, and calcium has a valence of 2. The amount of calcium carbonate required to replace the 0.0127 lb. of hydrogen then becomes $100 \div 2 \times 0.0127 = 0.64$ lb.

It was shown in Chap. IV that in the absence of leaching, but 26.9 per cent of the total available hydrogen would be exchangeable for calcium ions. Under this condition but 0.17 lb. of calcium carbonate per square yard of the road surface referred to above would be required. However, inability to control the conditions under which base exchange occurs in road surfaces and bases, in a manner comparable to laboratory control, requires the amount of neutralizer to be considerably in excess of that indicated by the computations.

Primes and Fillers Used to Increase Soil Stability.—Materials that effect a beneficial change in the electrical fields surrounding the surfaces of the mineral constituents may be used to advantage as primes when the natural soils have greater attraction for films of air, water, or natural gels than for those of the admixtures expected to provide stability. Particularly in the stabilization of such materials as the "blow" sands of Nebraska and the beach sands of Florida is it likely that films of natural gels would merit special consideration.

The effect of such primes on soils is similar to that produced by coating a metal with acid before the application of solder or shel-lacking the knots before painting lumber.

Winterkorn (131), Reagel, and Schappler (132) have experimented with soap primes as means to increase the adhesion between soils and bituminous materials. The greater adhesion of the oil for the soil provided by the soap pretreatment was demonstrated when a heavy rain washed the oil from a commonly constructed earth-gravel-oil mixture but did not wash it from the soap-treated section. It was indicated also that different soaps may be desirable for different soils.

In 1932, Victor Nicholson (133) called attention to the effect of surface chemistry upon the stability of asphaltic pavements. He described one sheet-asphalt pavement in Chicago in which the

sand grains on the surface became partly white and bare of asphalt not long after construction, as contrasted with other pavements in which the asphalt adhered so tenaciously that they did not become white. The surface whitening occurred during the first summer after the pavement was laid and was as bad in places where there was no traffic as where automobiles traveled.

In 1927, A. W. Dow (134) stated that "if two pavement mixtures, alike in every way as to composition, voids, etc., are tested for stability, the one containing mineral aggregate with the highest adsorptive value will show the highest stability." In 1929 Prevost Hubbard (29) stated: "Indications point to the fact that whether or not adsorption is directly responsible, those sands with high adsorption values possess more pronounced stabilizing characteristics than those with relatively low adsorption values, so that the two may go hand in hand."

Included among the filler materials which Nicholson lists as having greater affinity for water than for asphalt are: impure silica, gypsum, impure limestone, and common clay. Among those with greater affinity for asphalt than water are pure limestone dust and hydrated lime.

Present indications are that both calcium chloride and common salt are retained longer in road mixtures containing limestone or slag. This suggests that, as in the case of bituminous binders, the lime materials have greater attraction for the chemical admixtures than the silica soils.

There is evidence that the fillers having the greatest attraction for the stabilizing admixtures tend to provide the entire mineral mixture with properties producing a stronger bond with the binder. In this respect fillers may perform an electrochemical as well as a mechanical function.

As early as 1910 Prevost Hubbard (29) explained the effect of rock powders for increasing the bond in stone mixtures on the basis of gelatinous colloidal phenomena. Quotations from his book "Dust Preventives and Road Binders" on this subject are as follows:

In another paper by Cushman (135) on "The Effect of Water on Rock Powders" the following conclusions are reached: [a] "When water comes in contact with most rock powders, immediate reactions take place, which are to a certain extent analogous to those which take place with

cement and powdered glass. [b] The microscope reveals an accumulation of amorphous material of a gummy appearance largely associated with the surfaces of the crystalline particles as the action of water proceeds. [c] The effect of wet grinding is to increase the binding power or the cementing value of rock powders, and there are indications that the addition of small amounts of suitable electrolytes [soluble inorganic salts, acids, and bases] to the water will still further increase the action."

The last fact has a decided bearing upon the effect of blending different roadstones one of which through being partially soluble is capable of reacting upon the other to produce binding films. It has been noticed in cases in which macadam roads were being constructed of hard material, such as granite or diabase, which are difficult to bond under the roller, that the surface quickly compacted and gave satisfactory results when treated with a top dressing of limestone screenings. This observation led Cushman and Hubbard (136) to determine the cementing value of mixtures of these rocks with limestone as compared with the cementing values of the individual rocks. The results of a number of tests are given below and show conclusively that the addition of limestone to a feldspathic rock increases the binding power.

RESULTS OF TESTS OF THE CEMENTING VALUE OF GRANITE MIXED WITH LIMESTONE

Serial number		Cementing value		
Granite	Limestone	Granite	Limestone	Mixture
1431	1391	3	27	110
1432	1342	9	22	56
1435	1335	7	26	38
1435	1423	7	26	53
1574	1411	6	20	82

As the binding power of rock dusts is due to the decomposition or hydrolysis brought about by the action of water, it would follow that if this binding power can be increased by the addition of limestone, it is caused by further decomposition of the material, brought about by the interaction of calcium hydroxide, $\text{Ca}(\text{OH})_2$, resulting from the hydrolysis of the limestone particles. This is demonstrated by the following results obtained by determining the cementing value of a number of granites when treated with a small quantity of limewater, or calcium hydroxide solution.

These results show in every case a considerable increase in the cementing value of granites so treated and would indicate that the

RESULTS OF TESTS OF THE CEMENTING VALUE OF GRANITE MIXED WITH LIMEWATER

Serial number	Cementing value		Serial number	Cementing value	
	Alone	With limewater		Alone	With limewater
810	12	21	1,276	11	31
811	6	16	1,329	10	44
817	11	21	1,398	6	18
893	12	16	1,431	3	19
1,008	35	45	1,432	9	12
1,192	14	27	1,435	7	15
1,275	16	39	1,574	6	11

addition of a small amount of lime might greatly improve the binding value of certain roadstones. Of course the addition of a sufficient quantity of lime would produce a mortar in which the bond due to the crystallization of calcium carbonate will cover up any actual increase in the cementing value of the stone treated. This effect will also be produced to some extent when even a small quantity of lime is employed, but results given by Lord (137), who applied the principles developed by Cushman for rock powders, show that when a sample of chert and one of clinker were so treated, the increase in cementing value was greatly in excess of that produced by treating a chemically inert slag in a similar manner. These results are given below and indicate that the increase in cementing value is due to the formation of a hydrated silicate of lime.

Material	Mineral composition	Cementing value				
		Alone	With 1% CaO	With 4% CaO	With 8% CaO	With 13% CaO
Slag	Olivine and gehlenite	8	15	33	52	95
Chert	Amorphous quartz	6	9	22	106	2,000+
Clinker	Acid silicate	4	24	60	...	1,400

Requirements of Graded Materials.—The abrasive resistance and the degree of interlocking of soil mixtures are indicated by the grading of the granular fractions. Materials falling within the following limits, by weight, satisfy the foregoing requirements and should produce good surfacing layers.

Passing Sieve	Per Cent
1 in.	100
$\frac{3}{4}$ in.	85 to 100
No. 4.	55 to 85
No. 10.	40 to 65
No. 40.	25 to 50
No. 200 ¹	10 to 25

¹ The smallest fraction of the grading determination has been referred to at times as that passing the No. 270 sieve. The hydrometer method of analysis has been used to determine the fraction referred to as that passing the No. 270 sieve because that method is more practical than one utilizing a sieve as fine as the No. 270. However, the use of the No. 200 sieve is practical. Furthermore, it has been found that a knowledge of the fraction passing the No. 200 sieve serves our purpose as well as a knowledge of the fraction passing the finer sieve and eliminates the necessity of making the hydrometer analysis. This explains the substitution of the No. 200 for the No. 270 sieve in the grading requirements.

Material larger than 1 in. can be used under certain conditions, but the amount should not exceed 10 per cent. Also, the maximum size should never exceed one-third the thickness of the stabilized layer. The fraction passing the No. 200 sieve should be less than two-thirds of the fraction passing the No. 40 sieve.

For base courses the fraction passing the No. 200 sieve should be within the range of 0 to 25 per cent and should be less than one-half of the fraction passing the No. 40 sieve.

The liquid-limit and plastic-limit tests performed on the fraction of soil passing the No. 40 sieve are used to determine the relative capillary and cohesive properties of soils to be used as binders.

Generally, plasticity indexes of about 3 or less indicate sufficient binder cohesion for soil-road surfaces to be constructed on locations subject to unusually wet conditions; 4 to about 8 for conditions of average moisture; and 9 to 15, inclusive, only for the drier or the arid conditions. Plasticity indexes exceeding 15 indicate soils not suitable for this type of construction.

The presence of the undesirable micaceous, diatomaceous, peaty, or other organic substances is indicated by liquid limits greater than those indicated by the expression

$$LL = 1.6PI + 14$$

The more the liquid limits exceed such values the more unsatisfactory the soil binder is apt to be owing to detrimental sponginess

and capillarity. Elimination of such properties in detrimental amount from the final road-surfacing mixture may be accomplished by keeping the liquid limits below 35.

For base courses the material passing the No. 40 sieve should have a plasticity index of not more than 6 and a liquid limit of not more than 25.

The plasticity index and the liquid limit should be determined from tests conducted in accordance with standard American Association of State Highway Officials (A.A.S.H.O.) and A.S.T.M. procedures (138, 139).

Proportioning of Materials (140).—The percentage of each of several materials, as, for example, bank-run gravel, crushed stone or slag, soil fines, road metal, etc., required to furnish a mixture of a desired grading may be determined graphically by use of the trilinear chart shown in Fig. 124. Every point located within the triangle represents a definite mixture of the three soil fractions (*a*) coarse aggregate (retained on No. 10 sieve), (*b*) fine aggregate (passing No. 10 sieve, retained on the No. 200 sieve), and (*c*) soil binder (passing No. 200 sieve). Thus the point *A* represents a mixture of the following composition:

	Per Cent
Coarse aggregate.....	20
Fine aggregate	70
Soil binder.....	10

For the purpose of illustrating how such determinations may be made, assume the following data on samples of materials from three different sources:

Sample	A	B	C
Coarse aggregate, per cent.....	20	100	10
Fine aggregate, per cent.....	70	0	40
Soil binder, per cent.....	10	0	50
Total.....	100	100	100
Per cent passing No. 40 sieve (soil fines).....	40	0	60
Plasticity index.....	3	0	24

It will be noted that the grading represented by point *A* is that of sample *A*. The grading of sample *C* is represented by point *C*.

By varying the proportions of samples *A* and *C*, every grading represented by points on a line connecting points *A* and *C* may be obtained. If the desired grading is not represented by a point that lies on, or close to, this line, a third material must be included in the mixture.

To illustrate, let it be assumed that a mixture consisting of coarse aggregate, 50 per cent; fine aggregate, 35 per cent; and soil binder, 15 per cent, is desired. This grading is shown as

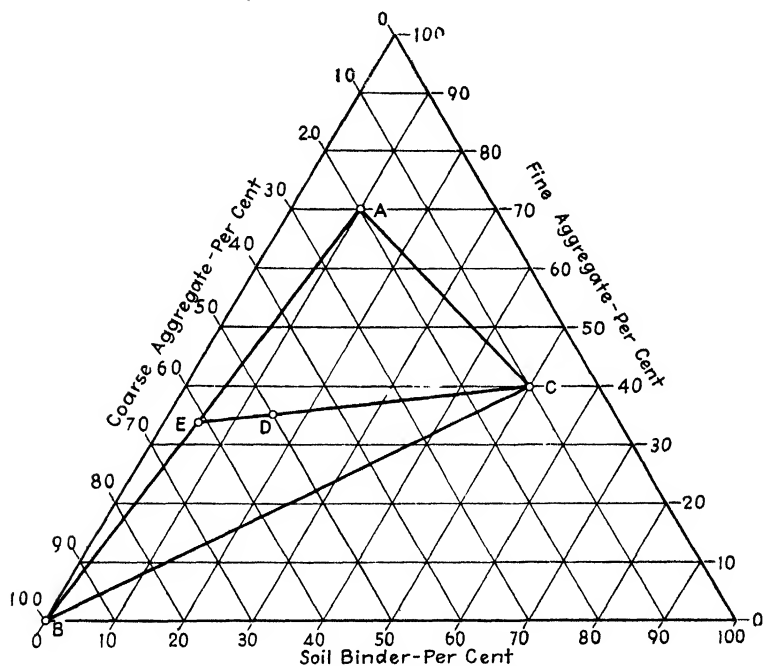


FIG. 124.—Chart for determining proportions of different materials in stabilized soil mixtures.

point *D* on the chart. The grading of the third constituent must be such that the point *D* will fall within the triangle formed by connecting the points representative of the gradings of the three materials to be combined. The grading of sample *B* satisfies this requirement and is plotted as point *B*.

A line drawn from *C* through *D* intersects line *AB* at point *E*. Since the points representing all possible combinations of samples *A* and *B* must fall on line *AB*, it is evident that a mixture of *A*

and *B* represented by point *E* is suitable for combining with sample *C* to give the composition indicated by point *D*.

The proportions of materials *A* and *B* required to produce composition *E* are represented by the proportionate lengths of lines *AE* and *EB*, expressed as percentages of the length of line *AB*. These percentages are, respectively, 51.6 and 48.4. A mixture of 51.6 per cent of sample *B* and 48.4 per cent of sample *A* will therefore produce composition *E*. It should be noted that the proportion of sample *B* in mixture *E* is represented by the proportionate length of line *AE*, or the end of *AB* farthest away from point *B*. Similarly, the proportion of *A* is represented by line *EB*.

The proportions of sample *C* and mixture *E* required to produce the desired final mixture *D* are represented by the proportionate lengths of *ED* and *DC*.

The proportions so indicated are sample *C*, 22.2 per cent; mixture *E*, 77.8 per cent. Since mixture *E* is composed of 48.4 per cent of sample *A* and 51.6 per cent of sample *B*, the percentages of samples *A* and *B* in the final mixture *D* will be, respectively, 77.8 per cent of 48.4 = 37.7 and 77.8 per cent of 51.6 = 40.1.

The final mixture, then, will consist of:

Sample	Per Cent
<i>A</i>	37.7
<i>B</i>	40.1
<i>C</i>	22.2

The gradation analysis of the soil mixture should meet the specifications given on page 269. It may be calculated from the sieve analysis and the percentage of each material used in the soil mixture, or it may be actually determined on the soil mixture. If the gradation analysis does not meet the specified requirements, further adjustment should be made in the proportions of the materials making up the soil mixture. If the requirements cannot be met by this means, other sources of materials should be sought.

A rough approximation of the plasticity index of a mixture made by combining materials from different sources may be calculated on the basis of the straight-line relationship between the plasticity index and percentage of admixture shown in Fig. 112, as follows:

Let X = percentage of sample A in mixture.

Y = percentage of sample B in mixture.

Z = percentage of sample C in mixture.

S_1 = percentage of soil fines in sample A .

S_2 = percentage of soil fines in sample B .

S_3 = percentage of soil fines in sample C .

P_1 = plasticity index of soil fines in sample A .

P_2 = plasticity index of soil fines in sample B .

P_3 = plasticity index of soil fines in sample C .

P = plasticity index of soil fines in mixture.

Then

$$P = \frac{P_1 S_1 X + P_2 S_2 Y + P_3 S_3 Z}{S_1 X + S_2 Y + S_3 Z} \quad (72)$$

As an illustration of this calculation consider the data given on pages 270 and 272, where

$$\begin{array}{lll} X = 37.7 \text{ per cent.} & S_1 = 40.0 \text{ per cent.} & P_1 = 3. \\ Y = 40.1 \text{ per cent.} & S_2 = 0.0 \text{ per cent.} & P_2 = 0. \\ Z = 22.2 \text{ per cent.} & S_3 = 60.0 \text{ per cent.} & P_3 = 24. \end{array}$$

Then

$$\begin{aligned} P &= \frac{37.7 \times 40.0 \times 3. + 40.1 \times 0.0 \times 0 + 22.2 \times 60.0 \times 24}{37.7 \times 40.0 + 40.1 \times 0.0 + 22.2 \times 60.0} \\ &= \frac{36,492}{2,840} = 12.8 = \text{plasticity index of mixture} \end{aligned}$$

This calculated figure should be checked by an actual determination of the plasticity index on the soil fines of the mixture. If the result does not fall within the desired limits, the proportions of the several materials making up the mixture should be further adjusted so as to bring the plasticity index within these limits and at the same time keep the gradation analysis within the specified limits. If this cannot be done, other sources of materials should be sought.

Function of Deliquescent Chemicals and Soluble Binders. The effect of traffic on well-graded topsoil, gravel, and similar roads, under proper conditions of maintenance, is to increase their density, provided the surfaces are kept slightly moist. When such surfaces are dry, the binder powders under traffic and not only forms dust but also permits the raveling of the surface,

which leads to surface roughness and eventually to complete disintegration.

If, however, such surfaces can be maintained in a damp or slightly moist state, the binder will prevent the separation of the granular particles, and the shocks and blows produced by traffic wheels then become effective in wedging the granular fragments into close association. Thus, the coarser stone, the sand, and the binder are formed into hard road surfaces. Binder clay present in amount only slightly in excess of that required to coat the sand grains is gradually forced to the surface of the roads as the granular fragments consolidate, especially during wet weather, and is gradually carried off in the form of dust or worked to the sides of the road by traffic. In this way the gradings required for the high stability may be automatically attained.

The importance of maintaining the proper amount of moisture in the surface cannot be overstressed. Absence of moisture from soil-road surfaces causes dust and raveling. Too much moisture causes rutting. The drier the road surfaces become between rains the wetter they become when it does rain. This is because extreme dryness causes small cracks to form in the clay binder through which rain water may enter and soften the interior of the road surface. Fissures do not form in damp surfaces of properly graded roads. Consequently water is shed from them without injurious effect.

It is the function of the deliquescent chemicals to assist in maintaining moisture in the surface. Calcium chloride is sometimes used for this purpose. Magnesium chloride might also be of benefit. Under certain conditions of humidity and temperature such materials have the property of absorbing moisture from the air and slowing up the rate at which soils lose moisture between rains. Under certain other conditions they have the property of releasing their chemically combined moisture to the surrounding soil.

Sodium chloride functions mainly to retain the moisture already present in the soil and as a soluble binder which acts as a cement owing to an interlacing of salt crystals formed in dry weather.

As the partially dried road continues to lose water, the solution of sodium chloride in the surface becomes concentrated to the saturation point and deposits fine crystals, forming a cohesive seal coat of salt and fine aggregate which appreciably retards further evaporation. Additional strength and stability is

obtained through the crystallization of sodium chloride within the pores of the compacted mass as evaporation slowly proceeds. This filling of the voids minimizes the shrinkage which commonly accompanies loss of moisture and thus reduces the tendency of shrinkage cracks to form. The importance of diminution of shrinkage in this way will be realized when it is considered that any shrinkage of the clay bond causes it either to crack or pull away from the coarse aggregates and thus allow the latter to become dislodged and "raveled" under traffic.

Treatment with either calcium chloride or common salt effects a decrease in the volume change and an increase in the density and the stability of graded road mixtures—the calcium chloride through electrolytic and deliquescent properties, and the sodium chloride through electrolytic and crystallization properties. Generally, the sodium chloride-treated surfaces are harder, with a dryer appearance and slightly more dust, than the calcium chloride-treated surfaces.

Because solutions of calcium and sodium chloride have lower vapor pressures than water has, the evaporation of moisture from soil mixtures wetted with these salt solutions is definitely slower than from similar mixtures moistened with water. These salts in a stabilized road mixture therefore tend to conserve its moisture content.

The value of this ability to conserve moisture is most apparent during compaction of the stabilized mat in which the salt has been incorporated. The plasticity of the clay component is maintained over a longer time, thus prolonging its period of seasoning during which compaction is taking place and permitting compression of the aggregate into a denser mass with thinner and consequently stronger and more lasting binding films than would otherwise be obtained.

Roads treated with calcium chloride or sodium chloride may become coated with a thin layer of mud during wet weather. However, it is believed that much of this is the result of an excess of soil binder or use of a binder having too much plasticity. Use of these chemicals will not prevent loss of stability due to faulty proportioning of soil materials in the mix. Proper grading of soil materials is an essential requirement in the stabilization procedures involving the use of either calcium chloride or sodium chloride. However, experience furnished by more than 150 miles of salt-treated roads and close to 2,000 miles of calcium chloride-

treated roads indicates that when used in connection with properly designed soil mixtures these chemicals have an important place in the highway field.

In the construction of soil base courses for either bituminous surface treatments or higher type wearing courses the use of chemicals may be of especial benefit. Soil mixtures for bases should have lower clay content than that required in stable surface courses. Compaction of such bases by traffic prior to the application of the surfacing is desirable. Only by the use of water-retentive chemicals can the base mixtures lean in binder be kept from dusting and raveling excessively under traffic.

Problems

1. What methods are used to accomplish the stabilization of soils?
2. What are the two methods of consolidating soil? Name the conditions under which each is suitable.
3. What is meant by the "optimum moisture content" of a soil?
4. Describe briefly the densification test for soils.
5. The results of densification tests performed on a sample of soil representative of the material used in an embankment are given in Fig. 120. The embankment was compacted at a moisture content of 22 per cent. What are the resulting density in pounds per cubic foot and the stability in pounds per square inch? Supposing that after construction the embankment should be covered by water due to a flood, what change might occur in the moisture content and stability?
6. Name the types of admixtures that may be used in the stabilization of soil.
7. What is the nature of the benefit provided by admixtures of (a) deliquescent chemicals, (b) electrolytes, (c) primes, (d) fillers, and (e) neutralizers?
8. What are the essential requirements of a stable soil mixture for (a) a road surface; (b) a base course?
9. The gradings of three soil materials are as follows:

Sample number	1	2	3
Percentage passing 1-in. sieve.....	100		
Percentage passing $\frac{3}{4}$ -in. sieve.....	99		
Percentage passing No. 4 sieve.....	76		
Percentage passing No. 10 sieve.....	31	100	100
Percentage passing No. 40 sieve.....	3	99	68
Percentage passing No. 200 sieve.....	1	88	11

What proportions of each should you use to produce (a) a stabilized soil road surface; and (b) a stabilized soil base course? What are the resulting gradings?

CHAPTER XIII

STABILIZATION OF FINE-GRAINED SOILS

Practice in the stabilization of fine-grained soils differs in several respects from the procedures used in the construction of roads with well-graded mixtures. Care must be taken to provide for the optimum moisture contents, when only water or electrolytes are used as admixtures, because maximum density can be obtained only under that condition.

Thorough Distribution of Binder Important.—Special means must be used to insure thorough distribution when cement and bituminous materials are used as binders and when crushed rock, slag, etc., are used as primes or neutralizers. This is because of the much greater surface area to be covered in the fine-grained materials.

Consolidating equipment designed to produce a predetermined degree of densification during construction must be used, instead of depending upon traffic to produce the maximum density afterward, although a slight increase in density due to traffic may occur under certain conditions. When only water or electrolytes are used as admixtures, it is especially important that the soils should be densified by means of a sheep's-foot roller, or apparatus giving similar effect, and at optimum moisture content and to a density indicated by the Proctor tests. Construction methods with admixtures of Portland cement and bituminous materials may not be so critical.

Finally, bituminous surface treatments are required to provide fine-grained soil roads with the resistance to abrasion and softening under traffic furnished by the coarse aggregate in graded soil mixtures.

Insoluble Binders.—In dams and other embankments water alone has been used as the admixture. Of the water-insoluble materials, Portland cement and bituminous materials are the admixtures that have been most experimented with in road construction up to the present time. Among other possibilities are dual treatments with sodium silicate and calcium chloride to

produce insoluble calcium silicate; also dual treatment with ferric chloride and ammonia to produce the highly adhesive ferric hydroxide.

In the type of stabilization in which insoluble adhesives are used the fact that the films of adhesives might be thicker than those of moisture alone, or moisture with electrolytes, at the same degree of soil compaction has no significance with respect to the relative stability of the soil with the different films. The use of

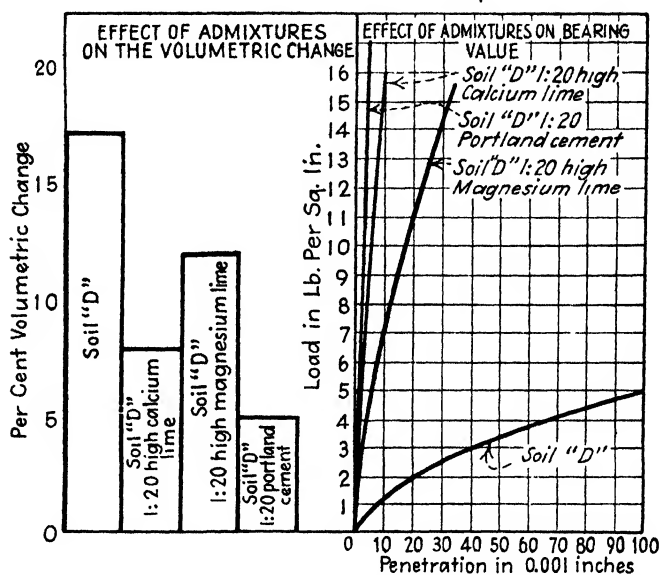


FIG. 125.—Effect of lime and cement on the shrinkage and bearing value of a soil.

insoluble adhesives is not expected or intended to render the soil sufficiently hard or tough to resist the abrasive action of traffic but simply to render it resistant to water from capillarity and thus to retain in the soil the bearing-strength characteristics which it has when in a dry compacted condition.

Portland Cement and Bituminous Binders.—In 1925 Golbeck reported in the *Transactions of the A.S.C.E.* (141) the results of laboratory experiments which showed that 5 per cent additions of hydrated lime and Portland cement have the effect of decreasing the volumetric change due to variations in moisture content and likewise of increasing the bearing value of plastic soils that

contain moisture up to the limit of their capillary moisture. The results of these tests are illustrated in Fig. 125.

Additional evidence that cohesive binders in soils are likely to prevent both their expansion and disintegration due to water absorption resulted from tests reported at the Eighth Annual Asphalt Paving Conference (142). Figure 126A shows six soil cakes representing soil existing at different depths in a subgrade located at Arlington, Va., treated with water-gas tar in 1923. These cakes were recompressed in a semidry state, dried to constant weight, and immersed in water for two weeks in 1929. The two cakes shown on the left contain tar in appreciable amount and exhibit no signs of disintegration. The third from the left contains but a small amount of tar and has crumbled to a slight



FIG 126A —Soil samples containing water-gas tar in varying amounts, photographed after two weeks' immersion in water

degree along the top edges. The fourth cake from the left contains but a slight trace of tar and shows crumbling in appreciable amount near the top. The two cakes on the right contain no visible trace of tar and have disintegrated to a marked extent.

Figure 126B shows the effect of Portland cement and bituminous material on a silt soil, as reported before the Twelfth Annual Meeting of the Highway Research Board (70), December, 1932. Sample 1 was untreated and slaked completely when immersed in water for a few minutes. Sample 2 was treated with Portland cement, and sample 3 with bituminous material. Both samples 2 and 3 showed no evidence of slaking during periods of immersion lasting several months.

Efforts to stabilize fine-grained soils by admixtures of lime and cement materials were made in Iowa and South Dakota as early as 1924 and in Ohio several years later. The results furnished by this early work were not particularly promising, and they should not be considered as indicating the possibilities of such treatments, because the requirements of thorough dis-

tribution of admixture, high degree of compaction, and protective surface treatment now deemed necessary were not recognized in the earlier work.

More recently research in the possibilities of the use of Portland cement for stabilizing soil bases was performed in South Carolina by the State Highway Department (143). The purpose of this work has been to develop a base material which could be constructed at less cost than that required to provide one of properly graded sand clay or topsoil.

Experimental work in the treatment of soils with bituminous materials has recently been conducted in various parts of the

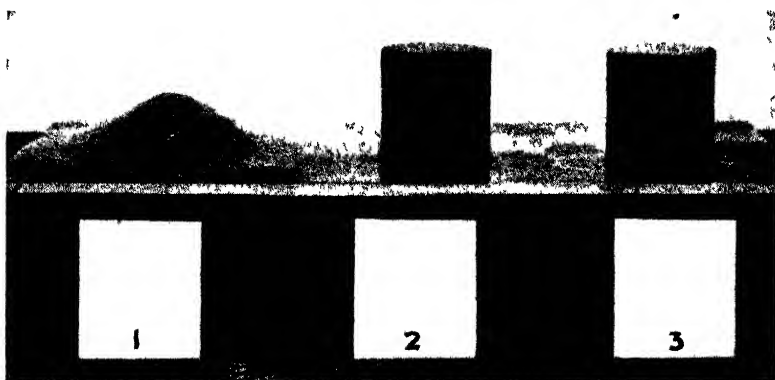


FIG 126B - Effect of Portland cement and bituminous material on the slaking properties of a silt soil

country. A number of highway projects have been constructed utilizing the different grades of tars, road oils, and bituminous emulsions. Airplane-landing fields have also been stabilized with bituminous materials.

Use of Other Adhesives. *Waste Sulphite Liquor.*—In the manufacture of wood pulp by the sulphite process the lignin and other encrusting matters of the wood fiber are broken down, resulting in a thin, light-brown waste liquid which, by evaporation, may be converted into a gummy residue. Products prepared from the crude liquor have been marketed from time to time under trade names such as Glutrin and Raylig, for use as binding mediums in road construction. As early as 1910 Hubbard (29) published results obtained by the Bureau of Public Roads, showing the increase in compressive strength obtained by

treating various rock powders with the liquors. Illustrative of the results obtained, it was found that the crushing strength of trap, gneiss, and sandstone rock powders would be increased from 300 or 400 lb. per square inch when water alone was used, to better than 2,000 lb. per square inch when the concentrated sulphite liquor was applied. However, the binding base is soluble in water, and as a result it was found that frequent rains tended to destroy the bond and remove the lignin from the road surface.

In order to offset this difficulty, attempts were made, according to Hubbard, at that time to incorporate the sulphite liquor in some material which, upon drying, would waterproof or make insoluble the residual base without destroying its binding value. The materials incorporated included semiasphalt road oils and deliquescent salts, such as calcium chloride. When subjected to favorable conditions it was concluded that concentrated waste sulphite liquors were in the nature of semipermanent binders suitable for use only upon water-bound macadam and similar roads. Within the past year waste sulphite liquor, marketed under the trade name of Raylig, has been experimented with by the State Highway Department of Washington for use primarily as a dust palliative, although the possibilities for its use as a stabilizing agent, especially in well-graded mixtures, has not been overlooked.

Molasses Residues.—In the manufacture of sugar from sugar cane, a thick, sirupy, liquid by-product, containing resinous and inorganic constituents, is obtained, which is known as blackstrap and which, when treated with quicklime, forms compounds of high binding value known as calcium sucates. Blackstrap, in combination with oil and lime, was applied as an experiment by the U. S. Bureau of Public Roads on a road at Newton, Mass., during the summer of 1908 with resulting indications that this combination had binding properties the permanency of which was not reported (29).

Additional information on the use of waste sugar products is contained in a discussion presented before the Indian Road Congress, held January, 1936, at Bangalore, India, by the chief engineer of the Mysore government, in whose district some interesting experiments have been carried out (144). Attention was called to a practice in India in the olden days when a kind of coarse sugar, made by evaporation from the sap of palm trees,

was added to lime to produce an improved quality of mortar. With this as a background, a short length of road surface formed of lime kunkur¹ was treated, and the results were encouraging. In order to prevent the molasses from being washed out during rains, slaked lime was mixed with it, the claim being that the addition of burnt lime to molasses produces tricalcium sucrate, which is insoluble in water.

Calcium Humate.—Dr. Winterkorn (145) has pointed out that the presence of organic matter and lime in an amount sufficient to coagulate the clay to particles of the size and character of silt increases the resistance of gravel roads to erosion. The flocculation, in this manner, of clay occurring naturally in the disperse state, causes a decrease in the effective clay and colloid content similar to that produced by the heat treatment referred to previously, but in this case may be a more reversible than permanent phenomenon.

The action of the lime material has been described by Russell (79) as being in principle very like that of any other salt. The calcium replaces hydrogen or sodium in the clay, thus converting an acid or alkaline clay into the more usual calcium clay; it also replaces the acidic hydrogen in humic acid, forming the more stable so-called calcium humate or neutral humus.

Calcium Silicate.—The insoluble calcium silicate binder is produced by treating soils first with sodium silicate followed by the application of calcium chloride according to the general principle used in the old English method of manufacturing artificial stone. Its use in soil stabilization has been principally in connection with increasing the stability of granular soils in connection with the construction of buildings and other structures.

Electrochemical Treatment.—In a method utilizing electricity for stabilizing soils and described by Endell and Hoffman (146) metal electrodes are introduced into the soil at appropriate distances, aluminum always being used for the anode, and copper for the cathode. Direct current is then allowed to pass between electrodes until the soil is hardened. Experiments were described in which a clay specimen had been brought into its container with an almost liquid consistency, the water content being about 80 per cent. After treatment for some time with a current of 8 to 14 amp., at 300 to 500 volts, the clay had become so hard that an

¹ A kind of limestone usually occurring as nodules in topsoil.

iron rod of 1 sq. cm. cross section failed to penetrate the clay when placed under a load of 10 kg. Specimens cut from the side of this block nearest the aluminum electrode neither disintegrated nor swelled during several months of immersion in water. The process has been patented.

Application of Heat (147).—The essential features of this method of treatment are illustrated by a method used in the so-called "black soil" areas in Australia, in localities where the haul for gravel or crushed rock is excessive. A slow-moving, downdraft furnace with a speed of about 10 ft. an hour was developed at Sydney, New South Wales, for baking the soil in place and converting it into a bricklike material. The machine

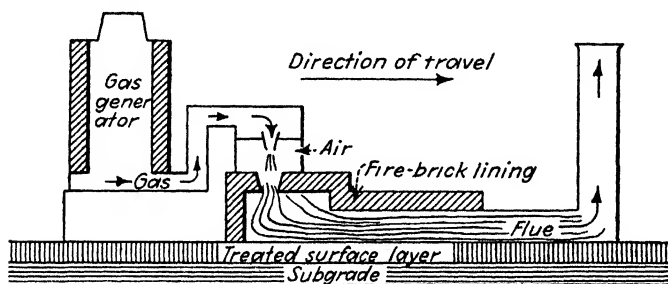


FIG. 127.—Diagram of furnace for heating soil in place.

(Fig. 127) is wood fired, of the air-gas producer type, on a chassis on road wheels, and is propelled under its own power. Prior to heating, the road is graded to the desired cross section, and the strip to be treated is scarified to a depth of about 4 in. As the machine advances over a particular area, the soil is first subjected to the exude of the gases and then to gradually increasing temperatures until the full intensity of the heat of the gases issuing from the generator is reached. Work proceeds continuously day and night, a 6-ft. wide machine completing about 500 ft. of roadway 18 ft. wide per week. The effect of the treatment is noticed to a depth of about 6 in.

When traversed by roller the treated clods are broken down fairly readily. The clay binder is applied to the broken material, the road shaped up and rolled again, and then the road is ready to be opened to traffic.

Surface Treatments on Densified Soils.—Prior to the advent of the automobile the principal purpose of the road surface was to

furnish freedom from mud throughout the year. The effect of horse-drawn traffic was always to compact loose, granular material into a firm, coherent, wearing course. This was because, except when brakes were applied, rolling-wheel tires always exerted a compacting influence similar to that furnished by a road roller.

With the advent of the self-propelled vehicle, new and destructive elements were introduced in the form of thrust and possibly suction produced by the driving rubber-tired wheels. This caused the mineral binder to become loose and the larger fragments of road metal to become separated, leading to general disintegration of the road surface. While the disintegration of the road surface was perhaps the more serious of the two evils, in so far as the roadbuilder was concerned, the clouds of dust raised by the motor car constituted the greater nuisance insofar as the traveling public was concerned. As a result, all kinds of bituminous materials, as well as the deliquescent chemicals noted previously, were used as surface treatments primarily as a means of abating the dust nuisance.

As our knowledge of roadbuilding and maintenance developed, it was gradually found out that bituminous surface treatments exert a stabilizing influence upon the structure of the wearing course besides increasing its resistance to abrasion. This is due to their waterproofing properties.

Two conditions are essential if the waterproofing of a soil surface is to be worth while. The materials of the surface and base must be either impervious enough or have sufficiently low capillarity to prevent the accumulation of enough capillary moisture to cause instability. The quality of materials necessary will be affected by climate, topography, and traffic. The surface treatment must be maintained sufficiently impervious to prevent the surface water from entering clay soils from above and to prevent the evaporation of the cohesive moisture films from sandy soils.

Stabilization by surface treatment is excellently illustrated by the blotter-type tar-and-asphalt surface treatments on the heavy gumbo-soil roads of western Minnesota and eastern North Dakota (148). Figure 128 shows a condition in the spring of a road west of Ada, Minn., prior to the first treatment in 1924. Gravel used for surfacing was found to have penetrated to depths as great as 3 ft. Figure 129 is a picture taken in the spring of the

year and shows the condition of a similar road after receiving a surface treatment of bituminous material with gravel covering. On inspection in 1932 the total thickness of the surface treatment was found to be slightly less than 1 in.



FIG 128 —Untreated gumbo-soil road in spring of year.

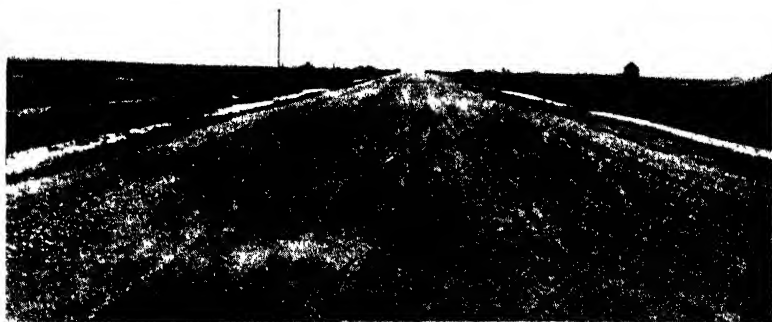


FIG. 129 —Gumbo-soil road with blotter-type bituminous-surface treatment, spring of year

Bituminous surface treatments have long been used in the Southern states to change loose, sandy and dusty materials into firm, stable, durable road surfaces. Prevention of evaporation accounts for at least part of this benefit. The maintenance of high density requires that loss of moisture from the densified soil by evaporation also be prevented.

Characteristics of Stabilized Soils.—The effect of compactive effort, temperature, and admixtures on stabilized soils has been determined by the Bureau of Public Roads (149). The gradings and physical properties of the soils used are indicated by the test data (Table 28).

The effects of moisture content, size of footing, depth of penetration, temperature, and clay content have been determined by means of the penetrometer at George Washington University (150). The physical characteristics are indicated by the data (Table 29).

Effect of Compactive Effort.—The data (Fig. 130) for the Manor soil show that the greater the number of blows of the

TABLE 28.—RESULTS OF TESTS ON DIFFERENT SOILS INVESTIGATED
BY BUREAU OF PUBLIC ROADS

Mechanical analysis ¹							
Soil	Particles larger than 2.0 mm.	Particles smaller than 2 mm., per cent by weight					
		Coarse sand 2.0 to 0.42 mm.	Fine sand 0.42 to 0.05 mm.	Silt 0.05 to 0.005 mm.	Clay smaller than 0.005 mm.	Colloids smaller than 0.001 mm.	Passing No. 40 sieve
Red clay	0	2	10	20	68	51	98
Iredell	0	11	11	20	58	42	89
Arlington	0	1	32	36	31	22	99
Manor	0	2	47	39	12	8	98

Physical characteristics¹ of material passing No. 40 sieve

Soil	Liquid limit	Plasticity index	Shrinkage		Moisture equivalent	
			Limit	Ratio	Centrifuge	Field
Red clay	65	47	10	2.0	31	25
Iredell	78	55	12	2.0	76 ²	40
Arlington	27	7	19	1.7	33	21
Manor	37	11	38	1.4	22	42

¹ Determined according to A.A.S H.O. and A.S.T.M. methods.

² Waterlogged.

TABLE 29.—GRADING AND PHYSICAL CONSTANTS OF SOILS INVESTIGATED¹ AT GEORGE WASHINGTON UNIVERSITY

Soil No.	Mechanical analysis, per cent						Physical characteristics of material passing No. 40 sieve					
	Particles larger than 2.0 mm.	Particles smaller than 2 mm					Liquid limit, per cent	Plasticity index	Shrinkage		Moisture equivalent	
		2.0 to 0.42 mm.	0.42 to 0.05 mm.	0.05 to 0.005 mm.	Smaller than 0.005 mm.	Smaller than 0.001 mm.			Limit, per cent	Ratio	Centrifuge, per cent	Field, per cent
1	0			Flocculated			32	9	19	1.8	31	25
2	0	6	38	12	44	33	55	30	24	1.6	32	36
3	0			Flocculated			40	0	48	1.1	20	45
4	0	18	82	0	0	0	21	0	29	1.5	3	27

¹ Data furnished by the U. S. Bureau of Public Roads

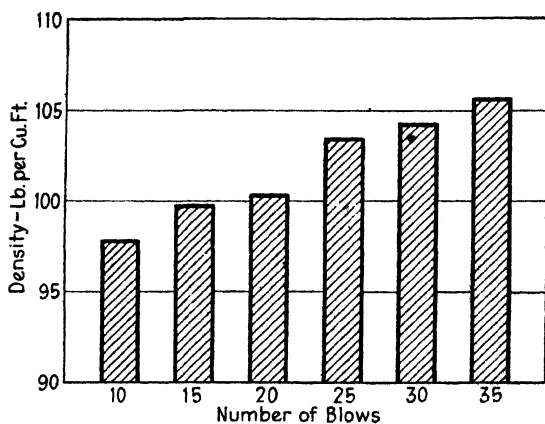


FIG. 130.—Effect of amount of compacting on the density of Manor soil.

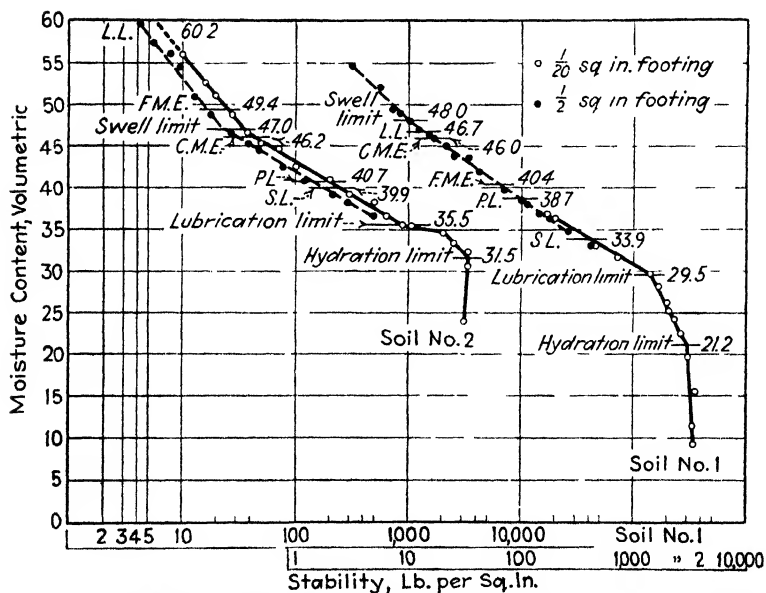


FIG. 131.—Relations between stability and moisture content in per cent by volume.

tamper used in compacting the samples the lower were the optimum moisture contents and the higher were the maximum densities, the range in density being 97.8 lb. per cubic foot at 10 blows to 105.6 lb. per cubic foot at 35 blows.

Effect of Moisture Content.—Data shown graphically in Fig. 131 reveal the relation of stability to moisture content. They consist of a series of logarithmic curves each of which is similar in form to that of the flow curves obtained with the liquid-limit device and described in Chap. V. They are expressed by the equation

$$W = A - B \log P \quad (73)$$

where W = moisture content, per cent by volume.

A = moisture content at pressure of 1 lb.

B = difference in moisture content corresponding to a difference of unity in the logarithm of the load P .

The stability-moisture content curves show that for the plastic soils, samples 1 and 2, the hydration, lubrication, and swell limits agree within the range of experimental error ($1\frac{1}{2}$ per cent moisture content) with these densification limits as disclosed by the density-moisture content relations of these samples as shown in Figs. 123*A* and 123*B*.

The curve (not shown) furnished by tests on sample 3—the cohesionless pumice—disclosed that the lubrication limit was approximately equal to the critical moisture content previously discussed in Chap. V (page 102).

As would be expected, a slight amount of moisture—1.4 per cent—added to the dry sand (sample 4) caused an abrupt increase in stability. With increase in moisture content from 1.4 to 12.0 per cent, bulking occurred with decrease in the density of the sample and was accompanied by a slight reduction in stability. With further increase in moisture content, bulking ceased, the density of the compacted sand increased, and the stability became greater until a moisture content of 26.7 per cent, the maximum that the sand would retain under conditions of free drainage, was reached.

Effect of Size of Bearing Area.—According to the data shown in Fig. 131, stabilities observed for the highly plastic soil—No. 2—above the critical moisture content were smaller when the plunger of $\frac{1}{2}$ sq. in. end area was used than when the plunger of the

smaller end area of $\frac{1}{20}$ sq. in. was used. For the feebly plastic soil—No. 1—the stabilities as indicated by the $\frac{1}{2}$ -in. plunger also were smaller than those indicated by the $\frac{1}{20}$ -in. plunger but only slightly so. For nonplastic soil—No. 3—the reverse was true, the larger stabilities being indicated by the plunger with the larger bearing area.

Only the *A* constant in Eq. (73) seems to have been influenced by the end area of the penetrating needle. The *B* factor, which is representative of the slope of the curve on the semilog plat, and also the moisture contents at which the breaks in the relationships occur, including those indicative of the critical moisture content, seem to be unaffected by the size of plunger used.

Data obtained but not shown indicate no relationship between stability and area of plunger for soils compacted below the critical moisture content.

Effect of Depth of Penetration.—Data on the effect of depth of penetration upon observed stabilities are shown in Fig. 132. The absence of uniformity of stability at different depths of the compacted samples is apparent. In the plastic soils—Nos. 1 and 2—the tendency for the pressure to build up with increase in the depth of penetration is considerably less than in the cohesionless pumice and beach sand—samples 3 and 4.

In order to determine the effect of the variation of stability with the depth of penetration, as indicated by the curves (Fig. 132), the stabilities observed at different depths were averaged, first, for depths of $\frac{1}{4}$ to 2 in. and, second, from 1 to 2 in., inclusive. In comparison with the maximum readings obtained throughout the range of $\frac{1}{4}$ to 2 in. depths, it was found that while the stabilities in the three instances vary somewhat for the different soils, the slopes of the curves are practically the same, and the indicated breaks occur at approximately the same moisture contents. Therefore, the maximum values obtained in a given depth of penetration seem suited for use in indicating the stability.

Effect of Temperature on Stability.—It is proverbial that molasses is "slower" in January than in June. The same can be said about water, for it also is more viscous in cold weather. Because of this, sedimentation proceeds at a slower rate, the films of hydrated water are thicker, for equal degree of compaction the moisture content of soil is greater, and for equal moisture content the stability of soil is greater during periods of low temperature.

It can be shown by the application of Stokes's law that the time required for settlement through a depth of 20 ft. of water at temperatures of about 100 and 35°F. would be, respectively, as follows: coarse-sand grain with a diameter of 1.0 mm., 4.6 and

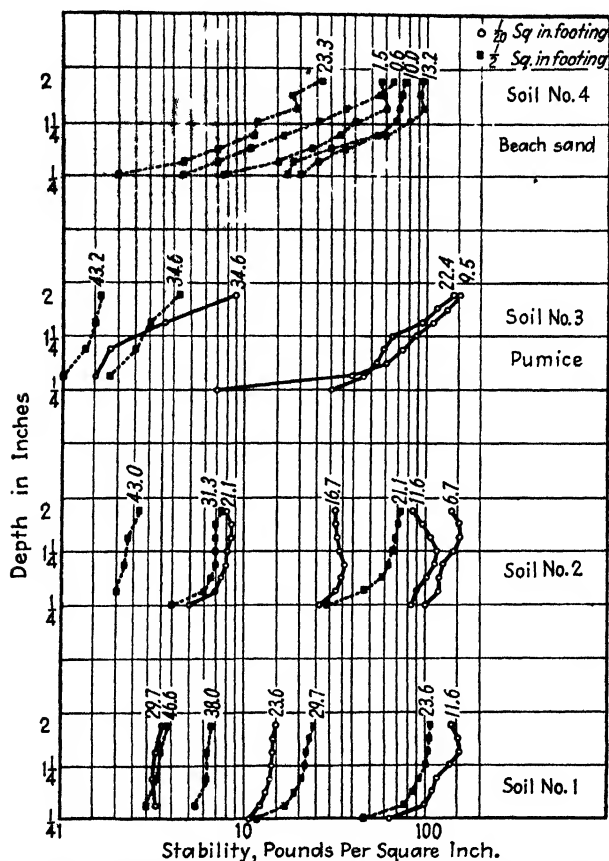


FIG. 132.—Stabilities indicated by different depths of penetration.

11.4 sec.; silt grains with diameters of 0.01 mm., 12.8 and 31.6 hr.; and clay particles with diameters of 0.001 mm., 53 and 132 days. In these computations only the increase in the viscosity of free water due to drop in temperature was considered.

The greater thickness of the adsorbed films is indicated by the data furnished by Baver and Winterkorn (78) and shown in

Table 30. Here, the cubic centimeters of adsorbed films per gram of colloid is greater at low than at higher temperatures. Thus the Putnam colloids ionized with sodium have 5.25 cc. of adsorbed film per gram at 30° and but 2.52 cc. at 99°C.

TABLE 30.—HYDRATION¹ OF PUTNAM AND WABASH COLLOIDS AS AFFECTED BY TEMPERATURE

Temperature, degrees centigrade	Hydration, cubic centimeters per gram of colloid ionized with .		
	Na	Ca	H
Putnam colloid			
30	5.25	9.75	5.20
50	4.56	3.90	3.52
70	2.53	1.98	2.16
99	2.52	0.86	2.70
Wabash colloid			
30	7.45	6.74	17.95
50	6.29	6.14	16.08
70	5.09	6.44	14.79
99	3.67	7.10	9.70

¹ From viscosities.

The data (Fig. 133) show that for the red clay the stability was 1,400 lb. per square inch when compacted at 42°F.; 960 lb. per square inch when the temperature of the sample was raised to 130°F.; 1,400 lb. per square inch when the temperature was then reduced to 33°F., and finally 1,100 lb. per square inch when the temperature was again raised to 118°F. This phenomenon is reversible.

Grading.—The effect of the different gradings shown in Fig. 134 and Table 31, on the densities obtained at equal compactive efforts and fairly constant temperatures, is disclosed by the data (Fig. 135). Curve 6 (Fig. 134) represents the grading of material passing the No. 10 sieve meeting the requirements of a good soil mortar.

The red clay (sample 1) contains 68 per cent clay, 20 per cent silt, and 12 per cent sand. At the optimum moisture content of

16.7 per cent, the dry weight was 113.0 lb. per cubic foot as compared with a dry weight of 129.8 lb. at 9.6 per cent optimum

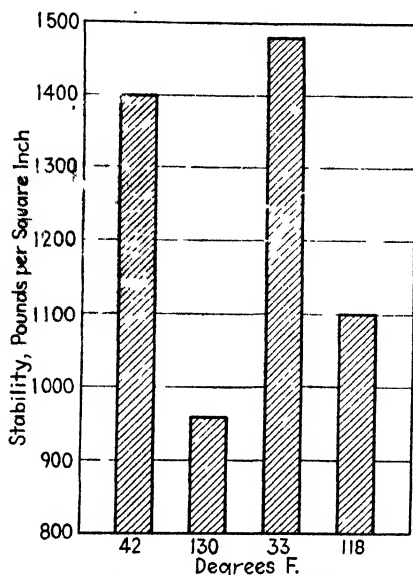


FIG. 133.—Effect of alternations of low and high temperature on the stability of red clay soil.

moisture content for the excellent soil mortar (group-A1 soil (sample 6)).

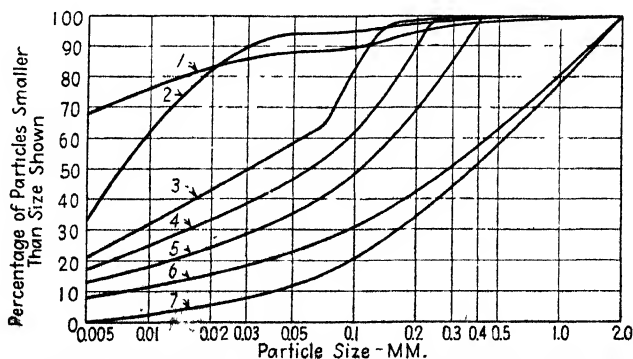


FIG. 134.—Grain size accumulation curves of sand-clay mixtures.

Samples 2, 3, 4, and 5 were obtained by adding silt and sand to the red clay represented by sample 1. An attempt was

made to obtain proportions representative of the different fractions of sample 6. Thus sample 5 represents approximately the grading of the fraction of sample 6 material that passes the No. 40 sieve; sample 4 represents the fraction that passes the No. 60 sieve; sample 3 represents the fraction that passes the No. 100 sieve; and sample 2 represents the fraction that passes the No. 270 sieve.

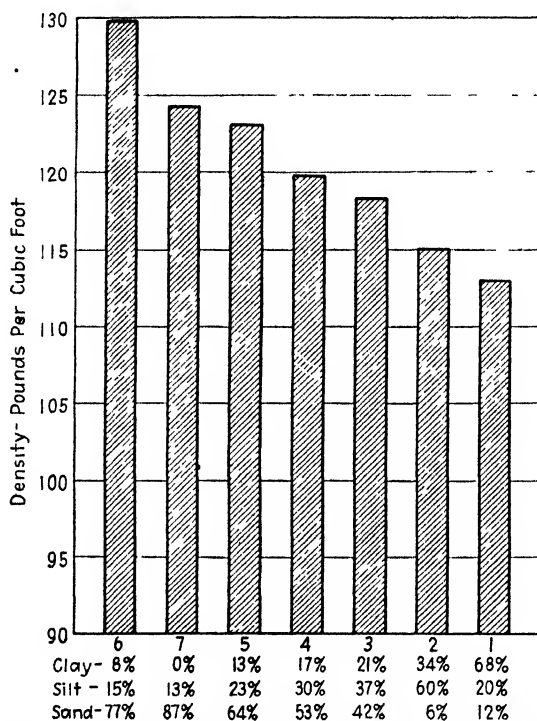


FIG. 135.—Effect of grading on the densities of compacted samples.

Figure 135 shows that as the total sand is decreased from that of sample 1 to that of sample 6, the density of the compacted mixtures decreases, and the optimum moisture contents increase. In sample 7 an attempt was made to approximate the grading of the fraction of sample 6 that was larger than 0.005 mm. As would be expected in the absence of the clay fraction, the optimum moisture content is smaller than that of sample 6, being only 8.5 per cent. The maximum density, however, is but

TABLE 31.—GRADING, OPTIMUM MOISTURE CONTENTS AND DENSITIES OF SAMPLES

Sample number	Mechanical analysis, per cent							Optimum moisture content, per cent	Dry weight at optimum moisture content, pounds per cubic foot
	Passing No. 10, retained on No. 20	Passing No. 20, retained on No. 40	Passing No. 40, retained on No. 60	Passing No. 60, retained on No. 100	Passing No. 100, retained on No. 140	Passing No. 140, retained on No. 270	Silt, per cent		
1	1	1	2	4	2	2	20	68	113.0
2	0	1	1	2	1	1	60	34	115.0
3	0	1	0	2	14	25	37	21	118.3
4	0	1	0	22	11	19	30	17	119.8
5	0	0	22	18	9	15	23	13	123.1
6	24	16	13	10	5	9	15	8	129.8
7	28	18	14	11	6	10	13	0	124.1
8	24	16	14	10	6	7	5	18	129.3
9	0	25	20	15	7	10	5	18	126.9
10	0	0	29	23	12	13	5	18	123.8
11	0	0	1	36	18	22	5	18	120.4

124.1 lb. per cubic foot, which is 5.7 lb. per cubic foot less than that of the excellently graded material (sample 6).

Sample 6 (Table 31) may be compared with samples 8, 9, 10, and 11 in which the sand, silt, and clay fractions are constant but the grading of the sand fraction is varied. The maximum density of sample 8 with 5 per cent silt and 18 per cent clay and approximately the same sand gradation as sample 6 is slightly lower than

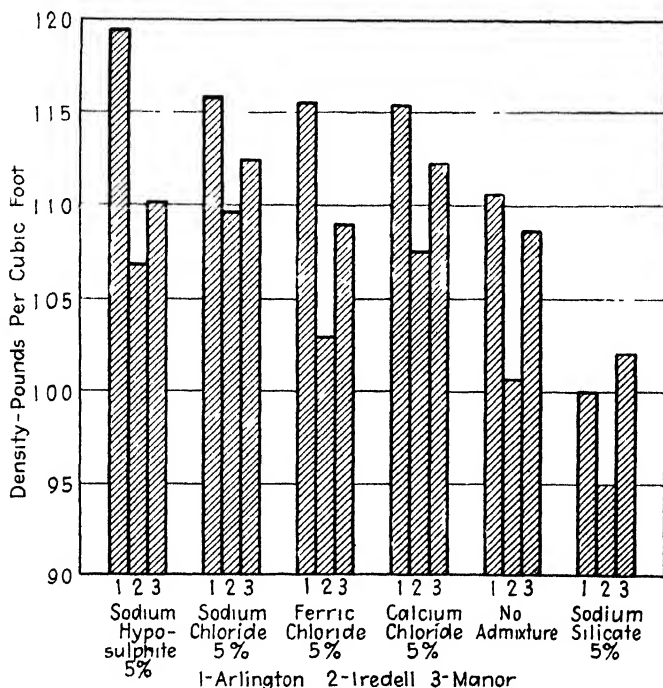


FIG. 136.—Effect of various electrolytes on the densities of compacted soils.

that of sample 6. As the coarser sand fractions are eliminated from samples 9, 10, and 11, and the mixtures approach a more nearly uniform grain size, their densities decrease.

Electrolytes.—In Fig. 136 and Table 32 are shown the effects of different electrolytes on the densities of compacted samples of the Arlington, Manor, and Iredell soils. Using the Arlington samples for illustration, it will be noted that admixtures of sodium hyposulphite, sodium chloride, ferric chloride, and calcium chloride effect an increase over that of the natural soil with water

alone, while the density of the sample treated with sodium silicate is less than that of the sample without admixtures.

TABLE 32.—EFFECT OF VARIOUS ADMIXTURES ON OPTIMUM MOISTURE CONTENTS AND DENSITIES

Soil	Arlington		Iredell		Manor	
Admixture	Optimum moisture content, per cent	Dry weight at optimum, lb. per cu. ft.	Optimum moisture content, per cent	Dry weight at optimum, lb. per cu. ft.	Optimum moisture content, per cent	Dry weight at optimum, lb. per cu. ft.
None.....	16.1	110.6	24.6	100.6	16.3	108.6
5% NaCl solution.	13.8	115.8	18.4	109.6	14.5	112.4
5% CaCl ₂ solution.....	14.0	115.4	20.8	107.5	15.2	112.2
5% Hypo.....	13.3	119.4	20.9	106.8	16.5	110.1
5% Hydrated lime.....	18.0	105.8	24.3	98.2	18.2	104.6
5% Soda ash.....	14.9	114.9	21.7	104.9	16.6	112.2
5% Sodium silicate.....			25.7	94.9	19.0	102.1
5% Ground slag.....	15.0	112.8	21.8	105.4	17.5	106.8
5% Ground limestone....	16.8	110.0	18.9	103.3	16.8	107.2
5% Ferric chloride	14.5	115.5	22.8	102.9	17.5	109.0
5% Raylig ¹	11.9	114.1	23.0	98.5	13.8	109.0
2% Tannic acid.....	15.4	110.3	24.1	97.7	16.5	106.0

¹ Waste sulphite liquor.

Fillers and Adhesives.—In Fig. 137 are shown the effects produced by other admixtures on the densities of different samples. The fact that the films of water-insoluble adhesives might be thicker than those of water has no significance with respect to relative stability of soils treated with different materials.

Static Pressures Produce Uniformly Compacted Samples.—If moistened samples are compacted by static loads instead of by tamping, moisture content-density relations such as are shown in Fig. 137A may be obtained for each soil. These curves show the optimum moisture contents, the hydration limits, and the lubrication limits for pressures ranging from 50 lb. per square inch to 1000 lb. per square inch. The densities at about 300 lb. per square inch pressure on this soil equal those furnished by the standard tamping method.

The relations of the optimum moisture contents to pressure, density, and stability, shown in Fig. 137B, are the control data for use in embankment construction.

For the compactive pressure to be provided by a given type of equipment the corresponding optimum moisture content,

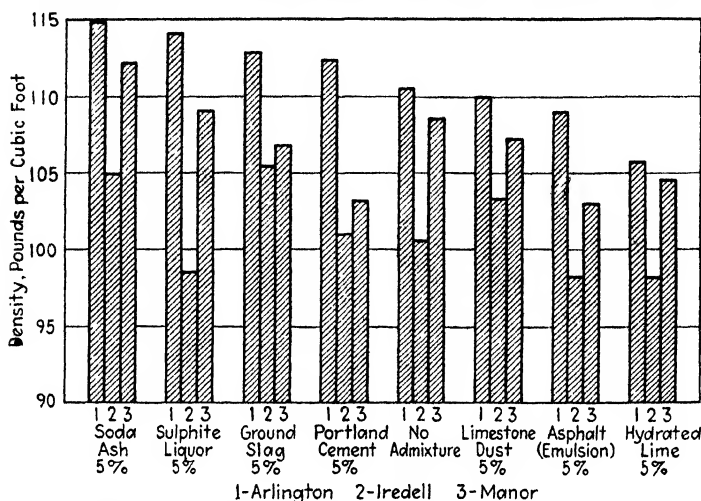


FIG. 137.—Effect of various adhesives and fillers on the densities of compacted soils.

resulting density, and plasticity needle readings are shown by the curves. Likewise for any desired embankment density, the equivalent pressure which must be furnished by the rolling equipment is disclosed.

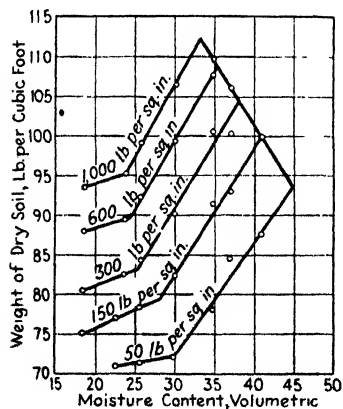


FIG. 137 A.—Density-moisture content relations for different static loads.

To illustrate, let it be assumed that the Proctor equipment, exerting a compactive effort equivalent to 300 lb. static pressure, is to be used. Then the soil, Fig. 137B, should be compacted at a moisture content of 38.2 per cent by volume (22.9 per cent by weight); the resulting density should be 104.4 lb. per cubic foot; and the plasticity needle reading, 117 lb. per square inch.

If, on the other hand, the embankment were to be constructed at an optimum moisture content equal to the plastic limit (41.4 per cent by volume), the soil

should be compacted at an equivalent static pressure of 130 lb. per square inch. The resulting density is 99.2 lb. per cubic foot, and the plasticity needle reading, 69 lb. per square inch.

Figure 137C illustrates how the control curves may serve to disclose the effect of soil character and admixtures on the densi-

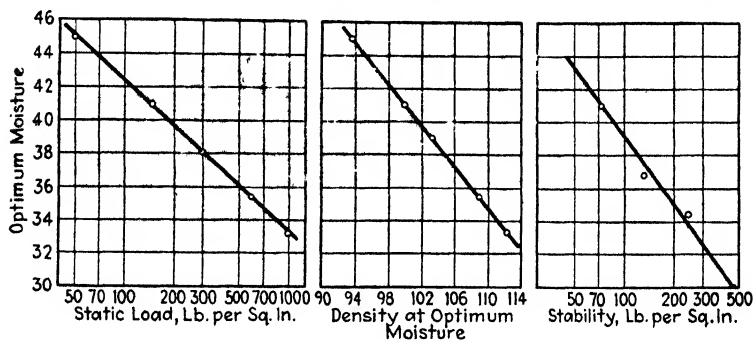


FIG. 137B.—Control curves, sample A.

fication of soils. Let it be assumed first that the full-line curves were furnished by tests of one kind of soil (A) and the broken-line curves by tests of another kind (B). The curves at the left show that at the same pressure of 135 lb. per square inch the optimum moisture content of soil A is 41.2 per cent as compared with but 33.0 per cent for soil B; also that to have the same

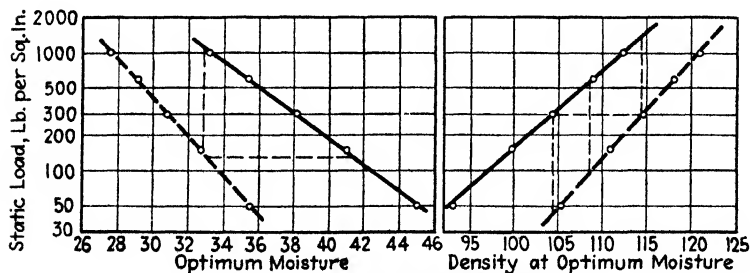


FIG. 137C.—Control curves, samples A and B.

optimum moisture content of 33.0 per cent, soil A requires a pressure of 1,100 lb. per square inch and soil B a pressure of but 135 lb. per square inch.

The curves to the right in Fig. 137C show that to attain the same density of 104.4 lb. per cubic foot, a pressure of 300 lb. per

square inch is required for sample *A* as compared with but 42 lb. per square inch for sample *B*; and that at the same pressure of 300 lb. per square inch soil *A* attains a density of 104.4 lb. per cubic foot, as compared with 114.5 lb. per cubic foot of soil *B*.

The control curves may be used in the same manner to determine the relative effect of (a) chemical composition of soil solids, (b) the kinds of ions adsorbed on the surfaces of the solid particles, and (c) admixtures of electrolytes, on the thicknesses of adhesive films which control stabilization. As a matter of fact both sets of curves, Fig. 137*C*, were obtained by tests of the same soil, the full lines showing the results with only water used as the admixture, and the broken lines showing the relations for the chemically treated soil.

Résumé on Stabilization.—Data on the effect of temperature, degree of compaction, grading, and admixtures indicate that

1. Fills compacted to a seemingly high degree of firmness may soften and lose stability when subjected to moisture, whereas fills compacted to a somewhat lesser degree of firmness may retain their stability under similar conditions of moisture. Also, the outcome is uncertain when embankments are constructed with a given degree of compaction without reference to moisture content or with a given moisture content without reference to the type or extent of compaction.

2. Soils compacted at low temperature may soften solely because of the water liberated by a rise in temperature.

3. For equal compaction greater densities can be attained at higher temperatures.

4. In order to obtain the same densities, more work of compaction will be required at lower temperatures.

5. At equal compaction, well-graded materials acquire the highest densities.

6. Admixtures of granular aggregate may effect a threefold change: improve the quality of the clay due to electrochemical phenomena, reduce the surface area, and increase the internal friction of the mixture. Those which provide for the strongest bond between clay and granular aggregate furnish the greatest benefit.

7. The lateritic types and the neutralized clays make the best binders. Inferior clays may be improved by admixtures of slag, crushed stone, gravel, chemicals, and the like.

8. According to Hubbard (29), mixtures of acid and basic rocks show a higher cementing value than either one alone. It is possible by selecting and blending certain roadstones or by treating the road with a suitable chemical salt or base greatly to increase the natural bond of the road surface.

9. The fact that mixtures have served satisfactorily as road surface is no assurance that they will not fail completely under the almost continuously

moist conditions produced when evaporation is stopped by the coverings of impervious wearing courses. Without a thorough knowledge of the swelling properties of the binder, the risk of failure which attends the practice of constructing impervious wearing courses on soil-bound mixtures which have become highly stable under traffic cannot be overemphasized.

10. It is feasible to utilize admixtures of electrolytes to facilitate attainment of desired densities when compacting the more porous soils and to decrease the effort required to consolidate soils of the better grade.

11. Any admixture that increases the thickness of the adhesive films will require a greater degree of compaction to attain the same density as that produced with moisture alone. In the case of admixtures whose effect is to reduce the thickness of the film, the same degree of compaction will produce higher densities with proportionate increases in adhesive strength and ultimate stability of the mixture.

The thinner the films the smaller is the amount of free water that can be released as the soil becomes warmer. Consequently, soils with chemically thinned films retain greater uniformity of stability under changing temperatures.

A second advantage is that reduced film thickness causes soil to attain high density with less compactive effort. This means lower cost for compacting fills, earth dams, and the like and less time required under equal traffic conditions for road mixtures to become stable. The latter is especially important in base-course construction where highest density is desired in the shortest possible time in order to prevent additional and perhaps nonuniform compaction of the base course, after the wearing surface is applied.

A third benefit has to do with frost action. The freezing point, as stated before, lowers as the thickness of the film diminishes. Therefore, the denser the soil at relatively low moisture content, the less likelihood is there of the films freezing even if they consist of water alone. When the contained chemicals also depress the freezing point, a double provision against damage to stabilized soil due to frost is obtained.

12. The lubricating properties of the mixing water or other admixture control the effort required to produce a given density. Its adhesive properties determine the stability of the consolidated soil. As a result, the compaction required to produce a given density increases considerably with change of the soil from the plastic to the semisolid state. The resulting stability, however, will be increased. As a practical consideration, therefore, it seems desirable to consolidate the soil in the plastic state but near enough to the plastic limit so that evaporation which occurs during the compacting period will be sufficient to leave the finally compacted soil in the semisolid state. However, as shown in Table 33, this does not mean that the plastic limit can be used as a substitute for the optimum moisture content to indicate the proper amount of mixing water required, although the optimum moisture contents of many plastic soils are very close to the plastic limits.

13. Many of the fine or poorly graded natural soils have greater attraction for the films of insoluble binder materials than for moisture, in which case

no trouble due to lack of adhesion between soil particles and binder need be expected. With soils high in silica, however, it is quite likely that a prime will be required. Use of certain soaps, Portland-cement washes, crushed rock, crushed slag, or mineral fillers may be of benefit.

14. Selected soil surfaces are suitable for temporary surfacing on important roads. They can be placed immediately after the grading operations for use during the period of settlement and will give substantial support and increased life to the pavements placed upon them. The method of stabilization by densification at optimum moisture content has been proved practical primarily in connection with the construction of embankments for use as earth dams, but it is equally applicable to any kind of fill or subgrade.

TABLE 33.—TABULATION SHOWING LACK OF AGREEMENT BETWEEN PLASTIC LIMITS AND OPTIMUM MOISTURE CONTENTS

Soil number	Plastic limit, per cent	Optimum moisture, per cent
8811	17	10
8701	20	16
8810	20	13
8687	23	25
8787	33	23
8814	33	20
8813	36	24
8816	36	24
8805	38	22
8815	50	30

15. In the construction of fine-grained base courses the use of the insoluble binders such as Portland cement and bituminous materials is likely to prove more satisfactory than attempts to stabilize with moisture films alone. Treatments with sodium silicate and calcium chloride in combination to produce calcium silicate precipitate also have possibilities for permanently changing the character of the soil.

16. The importation of commercial aggregates and fines to improve the quality of soils of the poorer grades may be advisable in the interest of ultimate economy of construction and maintenance and of type of service rendered.

The work with the improved penetrometer at George Washington University furnishes the following additional indications:

17. The moisture content-stability relations for the maximum and the average stabilities indicate that the effect of lack of uniformity in compaction may be largely overcome if a sufficient number of observations are made and to a depth greater than the thickness of each compacted layer of the sample.

Nevertheless, in research on basic soil physics, it seems desirable that a method of compaction be used that will produce more uniform density throughout the thickness of the sample than the method originally proposed by Proctor for use in field-control testing.

18. Observations should be made as nearly as possible at constant temperature.

19. The plastic limit as furnished by the Atterberg test obviously does not disclose the true plastic limit or critical moisture content of soils compacted in the manner used in the Proctor tests.

20. If observed stabilities are indicative of shear strength, then the assumptions that at the liquid limit all soils have equal shear strength and that for plastic soils between the liquid limits and plastic limits a constant logarithmic relation between moisture content and shear strength for each soil exists do not seem to hold true for compacted soils.

The more the stability depends upon internal friction, instead of cohesion, the greater seems to be the shear strength as indicated by the penetrometer at the liquid limit of soils. The stability of sample 1 (see Table 29), at the liquid limit is 4 lb. per square inch; of sample 2, 14 lb.; of sample 3, 36 lb.; and of sample 4, unlikely to be less than 52 lb., the stability under complete immersion. This indicates that the relative effect of shocks to produce flow in the liquid-limit test differs from the resistance to penetration in the stability tests depending upon the relative cohesion and internal friction of the soil.

21. Because of the effort required to make shear tests according to currently used procedures and the controversial aspect of proposed procedures for applying the results of such tests in practice, it would seem highly desirable to investigate the possibilities of using the simple penetration test just described as a substitute. With change of end area of the plunger, only the constant A of the moisture content-stability curves changes, being smaller and decreasing with increase in the size of the footing as the plasticity of the soil increases and being larger in nonplastic soils. Because of this, it is quite likely that by means of the stabilities determined with plungers of different end areas the relative amounts of cohesion and internal friction which furnish the total stability could be estimated.

22. One of the principal considerations in the selection of soils for compacted fills, dams, etc., is the amount of compactive effort required to produce the desired density, the greater amount of this effort required the less suitable being the soil, those requiring prohibitive effort being entirely unsuitable.

If high density is to be attained soil must be compacted at moisture contents above the hydration limit and as nearly as practicable at the lubrication limit. When the difference between the hydration and lubrication limits of a soil is smaller than the variation in moisture contents likely to occur under the best of control in the construction of embankments, the soil is obviously not suitable. This is true in the case of soil 2, Fig. 123B.

On the other hand, soil 1, Fig. 123A, represents material which was used satisfactorily in an earth dam constructed by the U. S. Forest Service near Gainesville, Ga., under the supervision of R. E. Pidgeon, Regional Engineer.

If the variation in moisture content of soil during embankment construction can be kept within about 5 per cent, a variation from $2\frac{1}{2}$ per cent below to $2\frac{1}{2}$ per cent above the optimum will occur when the soil is to be compacted at the optimum moisture content or lubrication limit as is the case when suitable materials are used.

Soil 1, the suitable material, has a density of 109 lb. per cubic foot at a moisture content $2\frac{1}{2}$ per cent below the optimum, 114.0 lb. per cubic foot at the optimum and 110.1 lb. per cu. ft. at $2\frac{1}{2}$ per cent above the optimum, making the maximum variation in density 5 lb. per cubic foot. Under the same conditions the densities of soil 2 are respectively 88.5, 106.5, and 101.5 lb. per cubic foot, causing the maximum variation to be 18 lb. per cubic foot.

If soil 1 is compacted to 109 lb. per cubic foot at $2\frac{1}{2}$ per cent moisture below the optimum, the stability will drop to 380 lb. per square inch at equivalent density above the optimum. The stability of soil 2 will, under the same conditions, drop to 54 lb. per square inch.

However, if the only material available were such as is represented by soil 2, it should be compacted at a moisture content above the optimum, the actual amount being determined from the slopes of the density curves in the lubrication and swell stages and at such moisture content that equal densities will result at the maximum variation of 5 per cent in moisture content.

In the case of soil 2, the moisture content which satisfies this requirement is 37.4 per cent instead of 35.7 per cent, the optimum. Under these conditions the densities will vary from 98.5 to 105.5 lb. per cubic foot, the minimum stability being 200 lb. per square inch.

23. The penetration tests seem to provide a means for designing graded mixtures on a quantitative basis, with stability as a criterion, rather than the qualitative tests for grading and plasticity. If the samples are prepared to represent as closely as possible the state of the mixtures in service, it is very probable that limiting stability values can be set up as a basis of satisfactory performance.

In order to determine the clay content for maximum stability under very wet conditions, graded granular material and clay binder available on particular jobs were combined in varying proportions and then placed in molds such as the Proctor compaction cylinder at moisture contents high enough to permit the mixture to slump to maximum density. The samples were then drained of free water and tested with the penetrometer. Figure 138 shows the type of data obtained. It will be noted that with only capillary moisture in the soil there is an optimum clay content—about 3.5 per cent (8 per cent based on material passing No. 40 sieve) in this case—at which the mixtures have maximum stability.

The combinations of any kinds of soil materials required to provide maximum stability at any moisture content or any degree of compaction can be determined by using the same testing procedure but varying the method of preparing the sample. Instead of testing at maximum capillary moisture, as noted above for base-course soil, samples of materials to be used for road surfacing may be tested at lower moisture contents. Data obtained to date indicate that the optimum clay content of mixtures increases as the moisture

content at which the samples are compacted decreases. This indicates the validity of the requirement that calls for variation of clay content with change in conditions of moisture under which the roads are to serve.

24. The control relations of a soil being gradually dried from a wet state may vary from those of the same soil when gradually wetted from a dry state. Also the data furnished by tests performed on a thoroughly pulverized material may vary somewhat from those furnished by tests of the same soil in which the natural structure has not been so thoroughly destroyed. Dried and powdered soils should be used only for laboratory research and identification tests. For control purposes in embankment construction the soil should be tested as it comes from the borrow pit, manipulated as necessitated by the laboratory technique, with the natural moisture content

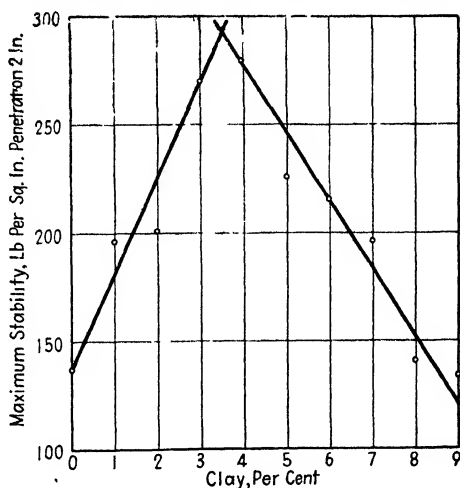


FIG. 138.

gradually increased and decreased. Otherwise the densities obtained by test are likely to differ from those produced by equivalent pressures during embankment construction. This fact was reported by Frank B. Campbell, *Engineering News Record*, January 30, 1936, and by C. A. Hogentogler, Jr., in the same publication, March 26, 1936.

25. Finally, there seems to be no consistent relation between any of the routine subgrade test constants and the moisture contents at which abrupt changes in the stability of compacted soils occur, with the possible exception that the critical moisture contents of nonplastic soils equal 75 per cent of their liquid limits. It should be emphasized that this does not affect the value of the routine tests for serving the purpose for which they are used—that of identifying dried and powdered subgrade soils according to types and fine-grained soils for use as binder in graded soil mixtures. For the latter purpose the liquid and plastic limit tests, especially, have proved by experience to be suitable and convenient tools.

For complete investigation of the properties of soils in compacted or otherwise stabilized state, however, a new set of constants equal to the moisture contents at which abrupt changes in stability occur, as disclosed by the penetrometer, seems well worth investigating.

Problems

1. What is the action of insoluble binders when used with soils?
2. Compare the action of the flat wheel-type roller with the tamping-type roller in the compaction (a) of fine-grained soils; (b) of coarse-grained soils.

CHAPTER XIV

CONSTRUCTION OF STABILIZED-SOIL ROADS

Construction of stabilized-soil roads is in the process of development. Numerous experiments are being carried on, some involving the use of the optimum moisture content and densification with sheep's-foot rollers, while in others traffic is depended upon for compaction. The best practices developed to date are outlined and can be used as a criterion.

Graded-soil Road Mixtures.—When the entire improvement is to be new, the required width is obtained by constructing a sufficiently wide roadbed or subgrade. Loose, sandy subgrades should be improved by the addition of soil binder; soft, unstable subgrades should be improved by the addition of granular material. The additions should be harrowed in, after which the surface of the subgrade may be trued up by blading and rolling. Subgrades consisting of firm soil can be trued to final grade and cross section by blading or otherwise cutting. Material of this kind, needed for binder in the wearing course, can be bladed to the shoulders of the road and left in windrows during the preparation of the base.

When an existing road is to be improved, the required width is obtained by excavating to a sufficient depth and width on each side of any existing gravel surface. The excavations are then filled in with the properly designed road mixture. The material excavated is used in building up the shoulders.

Where the soil in the existing surface or shoulders does not meet the requirements of stable materials, the deficiency is corrected by admixtures of gravel, stone, slag, sand, or soil binder as needed.

There are three methods for obtaining the desired mixture—plant mix, road mix, and stage construction.

Plant Mix.—The essential features of a plant-mix job are a source of satisfactory binder in close proximity to the supply of coarse aggregate which is to be used; means for drying and

pulverizing the binder; apparatus for measuring the quantities of binder and aggregate to secure the proper proportions; and equipment for thoroughly mixing the combined materials.

An ideal location for a mixing plant such as is shown in Fig. 139 is one at which the overburden consists of a clay soil having properties satisfactory for binder purposes. This overburden may be stripped and stock-piled in windrows so that it will dry sufficiently to be pulverized under a roller. The clay should then be placed in a bin and combined with an aggregate of proper



FIG. 139.—Mixing plant for preparation of stabilized-soil road materials.

grading in such proportions that the resulting mixture will conform with the desired specifications. Thorough mixing should be provided by means of a pugmill or rotary-type mixer.

In some instances, plant mixes have utilized bank-run gravel with the existing overburden and have eliminated the drying and pulverizing steps by passing the combined materials through a rotary screen which removes all over-size material (see Fig. 140).

The mixture as it comes from the plant is hauled to the road, spread, sprinkled, shaped, and rolled as described below. The principal advantages of the plant-mix method are greater uniformity and more thorough mixing of the binder and aggregate.

Road Mix.—The necessary equipment for mixing on the road includes tractor-drawn or self-propelled bladers, supplemented by scarifiers and apparatus for mixing, moistening, spreading, and compacting the base- and wearing-course materials. Scarifiers may be either of the four-wheel type or of the type attached to the roller (see Fig. 141).

Disk harrows should be such that unintended cutting into the subgrade can be avoided. Either the spring or spike-tooth harrows, which assure positive control of the height above the sub-

grade at which the teeth will work, are also satisfactory. For road mixing and shaping, power-drawn bladers with a wheel

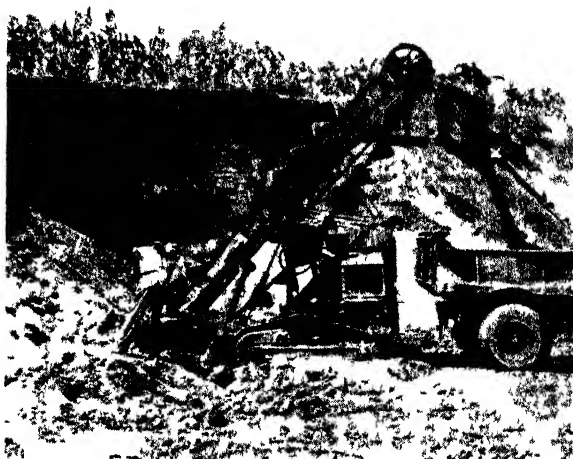


FIG 140 Mixing and loading gravel and clay overburden at pit

base of not less than 16 ft seem satisfactory (see Fig 142). Multiple-blade maintainers may be of the truck-suspended or -drawn type

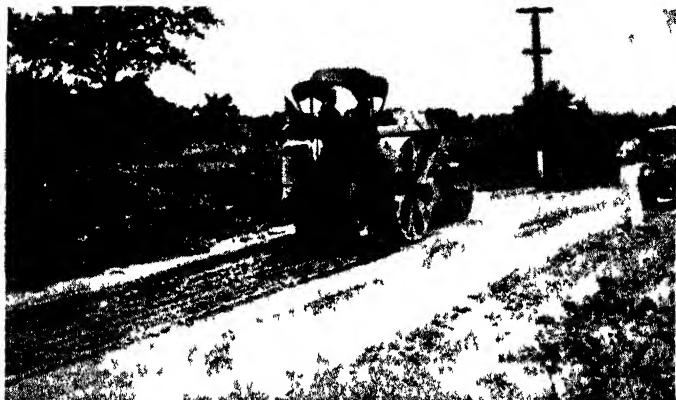


FIG 141 —Scarifying an old gravel road

The surfaces of existing roads should be scarified just deep enough to eliminate all irregularities of the surface and to permit

reshaping and adjustment to grade. After the loosened material is trued up to grade, the entire surface should be scarified so that if the loosened material were bladed away, a true base surface parallel to and at a required depth from the newly conditioned surface would result.

New material, if required, should be spread on the loosened surface in sufficient amount to furnish a compacted layer of approximately 3 in. Where greater surface thickness is desired, additional layers should be constructed.

Thorough working into a mixture of uniform properties throughout is then accomplished by means of harrowing and turning with bladers. If necessary, the mixture may be windrowed on the shoulders.

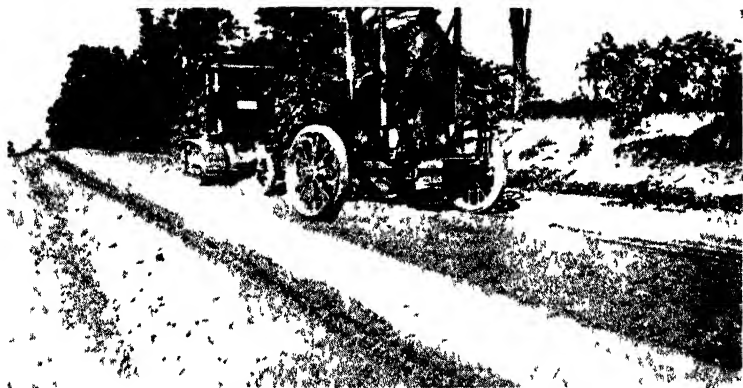


FIG. 142 — Mixing stabilized soil road material

The water required to furnish the proper moisture content in the wearing course is supplied, when needed, as follows: After the mixture of binder and gravel is bladed on to the shoulders of the road, the base is moistened uniformly, a suitable sprinkler (see Fig. 143) being used, and approximately one-fourth of the material from the windrows is bladed immediately on to the moistened base. The newly distributed road material is then similarly moistened and covered as before with one-fourth of the original windrowed gravel. These operations are repeated until the last layer of dry mixture has been placed.

Before moistening this top-surface material, the road is shaped and compacted. After shaping is completed the surface is thoroughly dampened and again compacted.

During the period of compaction the loose material should be kept distributed over the road surface by means of a road drag or blader. The crown of the road maintained by blading operations should be sufficient to provide for the rapid runoff of storm water in order to prevent the formation of pools of water on the road surface. For this purpose the center of the road should be maintained above the edges an amount equal to about $\frac{3}{4}$ in. per foot distance of the center from the edges. Pitting of the surface is an indication that the crown is too slight.

"Cut-and-try" Method.—In the construction of roads around farms, and from farms to adjoining local roads, or under similar

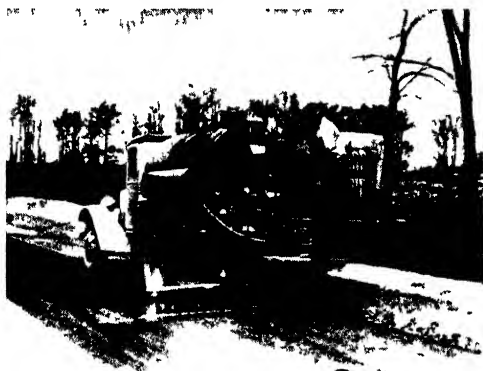


FIG. 143 —Sprinkling the subgrade preparatory to spreading the dry mixed material from the windrow

conditions where facilities for making the technical determinations may not be available, the proper grading to furnish stability must be determined by cut-and-try methods.

The cut-and-try method of road construction has been used extensively in the so-called "stage construction" and "traffic-bound" roads. Essentially it consists of adding granular materials to surfaces that become excessively muddy during wet weather and adding clay soil to loose, sandy and gravelly materials. The amount of either material added may vary considerably for different conditions and must be determined by trial. Usually, granular material is added in layers several inches thick. If after this material is worked into the soil a muddy condition persists in rainy weather, more material is added. If too much granular material is added, a loose condition of surface

develops. Special care must be taken to avoid adding an excess of clay to granular material. When material of a granular nature is added to clay the granular material may make up as much as 80 per cent of the total wearing surface, but a very small amount of clay, possibly only 15 per cent, will be required to stabilize loose sands and gravels. An effort should be made to have the clay powdered as much as possible.

A sample of well-graded sand and binder soil may assist in the selection of soil mixtures that have the desired properties.

If part of such a sample is wetted and squeezed in the hand, the following will be noted: (a) The soil is extremely gritty; (b) it can be formed into definite shapes which retain their forms even when dried; (c) if the clay alone adheres to the hands, it will be only enough to discolor them slightly; (d) if more than enough soil to discolor the hands adheres to them, it will consist of both sand and clay instead of the clay alone; (e) when the wetted sample is patted in the palm of the hand it will compact into a dense cake which cannot be penetrated readily with a stick the size of a lead pencil. These characteristics indicate well-graded material. The grittiness of the sample indicates the presence of sufficient granular material. Development of some strength on drying indicates a sufficient amount of binder soil. Resistance to the penetration of the pencil or stick, even when the sample is thoroughly wetted, indicates a desirable interlocking of the grains and the presence of a sufficient amount of capillary force.

Too much sand would cause the sample to fall apart when dried. Too much clay would leave the hand muddy after the wet sample was squeezed and would cause the wet sample after being patted to offer little resistance to the penetration of the stick.

Application of Calcium Chloride.—In plant mixes, the calcium chloride may be added and mixed at the plant or on the road. In road mixing, the addition of calcium chloride is accomplished in the following manner:

After the new and old soil materials have been spread over the prepared base in sufficient thickness, they should be covered uniformly with flake calcium chloride at the rate of $\frac{1}{2}$ lb. per square yard per inch of thickness, with a maximum of 2 lb. per square yard (see Fig. 144). The spreaders should be capable of delivering an even and uniform spread of the chemical in the

desired quantities. The calcium chloride is then thoroughly worked into the soil by harrowing and turning with bladers until the surfacing material has uniform properties throughout. Sprinkling and shaping into a finished surface are then accomplished in the same manner as in the case of the untreated roads just described.

In stage construction, calcium chloride may be applied to the road surface after its condition indicates that a proper gradation of materials has been secured. The chemical may be applied by mechanical spreaders, as in the case of road-mix jobs, or it can be conveniently spread by means of a shovel.

In periods of cold, wet weather the calcium chloride should be omitted during the mixing of the soil and subsequently applied to the surface of the road as weather and traffic conditions may require. The most favorable time for a surface application of calcium chloride is during the drying period just following a rain, after the surface has been bladed to proper smoothness. The moisture in the road at that time hastens solution and penetration. In the absence of rain, application in the early hours of the day, when the air has high relative humidity, is very desirable. Calcium chloride applied just prior to a rain will probably be washed off the road and consequently afford little or no benefit. When blading of the calcium chloride-treated road is required, it should be done after rains or begun near the end of a rain. At other times the surfaces may be too hard. For patching holes an admixture of 100 to 150 lb. of calcium chloride per cubic yard of graded soil is recommended.

Need for extensive patching during dry weather can be minimized or eliminated if close attention is given to the continuous maintenance of a moist road surface by the use of a number of light applications of calcium chloride during the season.



FIG 144.—Applying calcium chloride.

In the use of calcium chloride as a dust palliative, it has always been evident that this material remained effective for much longer periods in gravel having a clay binder than in gravel having a sandy filler. It has also been apparent that the presence of loose gravel tended to shorten the useful life of a treatment. Continual movement of such loose gravel under traffic and scraper blades exposes new surfaces to the air and promotes evaporation.

Application of Sodium Chloride.—Sodium chloride may be added at the plant in case of a plant mix or to the mixture on the road in the case of a road mix. The following suggestions from a project committee report issued by the Highway Research Board (151) call attention to important considerations in the use of sodium chloride:

Principally rock salt has been used, but any commercial type of salt or salt brine is satisfactory. The coarser grades of rock salt are well adapted for stabilization, as they remain free flowing and can be more uniformly spread than the finer gradations which will absorb moisture and cake when stocked along the road. Chemically there should be not less than 98 per cent pure sodium chloride.

When sodium chloride is applied in the form of brine, any of the gradations of the salt may be used, since the condition of the salt before it is dissolved is unimportant. The stabilized mixture will remain plastic longer if all of the salt has been dissolved, thus permitting rolling and compacting to be continued over a longer period of time with the possibility of attaining greater compaction at the time of construction. There should be enough water to moisten the soil somewhat beyond the plastic limit.

In Indiana it was found that from $\frac{3}{4}$ to $1\frac{1}{4}$ gal. of water per square yard per inch thickness was needed to moisten the mix satisfactorily. This was slightly more than 20 per cent by weight of the soil fines. In some cases it was possible to arrange the work so that the roads could be finished with the clay and aggregate moistened by rain. If there is overwetting of the surface mixture, there is danger of vehicles cutting through the surface course and mixing it into the subgrade.

Water is usually added by means of pressure distributors, sprinkler wagons, or gravity tank trucks. It is preferable to make several short trips over the course, sprinkling lightly on each

trip. This gives the moisture a chance to penetrate the entire course and does not result in a sloppy condition on the surface. The amount of salt used varies somewhat in the different states. The usual amount is about 2 lb. per square yard per 3-in. thickness of road constructed. Indiana requires $\frac{3}{4}$ lb. per square yard per inch thickness, regardless of thickness of the road constructed. A stabilized road treated with sodium chloride is shown in Fig. 145.

If during prolonged dry periods the fines wear off a sodium chloride-treated surface, leaving the larger aggregate particles protruding, the difficulty may be corrected by covering the surface



FIG 145 —Intersection of untreated soil road with stabilized road treated with sodium chloride

with properly proportioned soil mixtures containing about 100 lb. of sodium chloride per cubic yard to a depth of about $\frac{1}{2}$ in. When damp, the mixture makes a satisfactory bond with the worn surface without scarification.

If during periods of rainfall the top eighth or quarter inch of stabilized roads becomes plastic, about 50 to 70 tons of crushed aggregate per mile can be added to the surface. This should have a maximum size of about $\frac{1}{2}$ in. and should be free from dust and sand. Immediately after application, the material should be lightly floated over the surface and kept uniformly distributed as long as the surface is wet. The thin surface layer of fines acts as a cementing agent for the aggregate which has been added. This results in a satisfactory driving surface, adds some

additional thickness to the road, fills up any depressions that may have developed, and reduces the tendency for "potting."

Use of Portland Cement.—The construction methods employed in the South Carolina work (143) were substantially as follows:

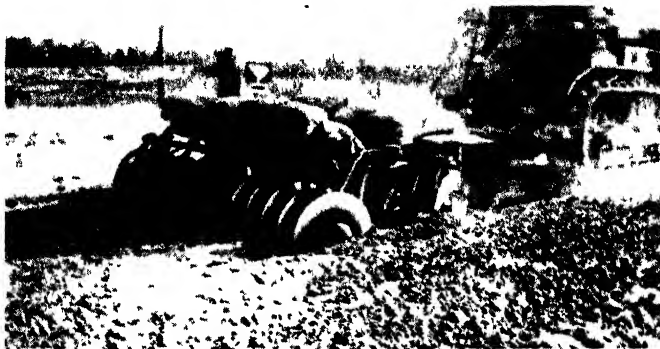


FIG. 146.—Mixing dry soil and Portland cement.

The cement was spread on top of the pulverized soil at the rate of about one sack per linear foot of roadway. Mixing of the dry soil and cement was done with disk harrows supplemented by a disk



FIG. 147.—Sprinkling mixture of soil and Portland cement.

plow or road machine (Fig. 146). Water was then added (Fig. 147) at the rate of about 8 gal. per linear foot, and compaction was accomplished by means of loaded trucks with dual tires

(Fig. 148). The surface was finished with a road machine after compaction. A bituminous surface treatment was applied subsequently to provide against surface abrasion.

According to W. H. Mills, Jr. (143), of the South Carolina Highway Department, all of the field experiments are in good condition at the present time, and no disintegration has been observed.

Bituminous Emulsions.—Current practice in the construction of stabilized-soil roads with bituminous emulsion is illustrated by a project in Solano County, California, which has been described by C. L. McKesson (152). Laboratory studies indicated that the natural adobe-clay soil could be stabilized to a



FIG. 148.—Compacting wetted mixture of soil and Portland cement.

depth of 3 in. with about 3 gal. of 55 per cent emulsified asphalt. By adding two parts of quarry waste to one part of the clay, stabilization was accomplished with 1 gal. of emulsion per square yard, a saving after paying for the quarry waste of about 5 cts. per square yard. The construction methods were as follows:

The quarry waste was uniformly spread on the clay subgrade to a loose depth of $2\frac{1}{4}$ in. and scarified into the natural clay. The loose mixture was $4\frac{1}{2}$ in. thick. The clay and quarry waste were well mixed by harrowing.

The soil mixture was then dampened with about 2 gal. per square yard of water, and the water mixed in by harrowing.

The emulsified asphalt was then applied, after dilution with four parts of water (Fig. 149).

The diluted emulsified asphalt was applied in six applications. The first spread was followed immediately with harrows, and then the first inch of the soil mixture and emulsion was bladed to the sides. This was repeated after the next two applications of diluted emulsion. The soil mixture was then in two windrows on the sides. It was then brought back over the road in three spreads, and dilute emulsion applied to each.

To secure thorough mixing, the soil-emulsion mixture was again bladed into windrows and then brought back in thin layers and finally shaped to the required cross section.

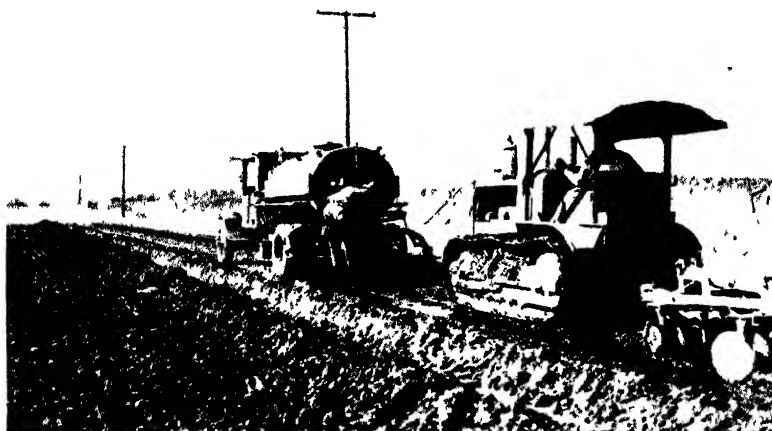


FIG. 149 —Applying bituminous emulsion and mixing with a disk cultivator.

The tractor used to pull the mixing equipment had by this time fairly well compacted the road surface. Figure 150 illustrates the compacting operation as performed on a similar job.

A total of 1 gal. per square yard of emulsion and 6.2 gal. of water was used, including the preliminary dampening operation.

This completed the soil stabilization, and only the surface treating remained.

The surface was then given an application of approximately $\frac{1}{4}$ gal. per square yard of the same emulsion, diluted with an equal part of water and immediately covered with 40 lb. per square yard of $\frac{1}{2}$ to $\frac{1}{4}$ in. crushed stone.

The surface of the road was then rolled. The crushed stone was embedded in the stabilized-soil mixture. A final applica-

tion of 0.64 gal. per square yard of undiluted emulsified asphalt was then spread and immediately followed with 15 lb. per square yard of $\frac{1}{4}$ in. to No. 10 sieve crushed stone, followed by brooming and rolling.

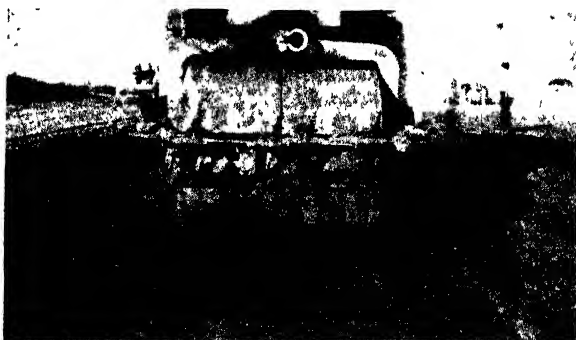


FIG. 150.—Compacting mixture of soil and bituminous emulsion.

The road was not closed to traffic at any time during the construction of this project. At completion it resembled bituminous macadam in appearance (see Fig. 151).

Construction of Portland-cement soil bases in South Carolina and emulsion-soil bases in Virginia in 1935 included the use of sheep's-foot tampers and controlled moisture content.



FIG. 151.—Bituminous emulsion stabilized-soil road with surface treatment.

Asphaltic Materials, Sub-oiling Method.—Work in Jackson County, Missouri, described by Frank Gilmore (153) is of the latter type. Asphaltic materials used as stabilizers were introduced at desired depths below the surface of the loose road mix

by means of a specially constructed "sub-oiler" illustrated in Fig. 152. It is essentially a toothed scarifier having an oil line attached to the back of each tooth, running nearly to its points.

The theory back of this procedure has been that as moisture evaporates from the overlying soil, the moisture films will be replaced by liquid asphalt. Whatever the reason, roads consisting of heavy, sticky clay or gumbo types of soil are water resistant after treatment and unaffected by rains.

Asphaltic Materials, Mixing Method.—A method of mixing the asphalt with the soil, used in Missouri during 1935, has been

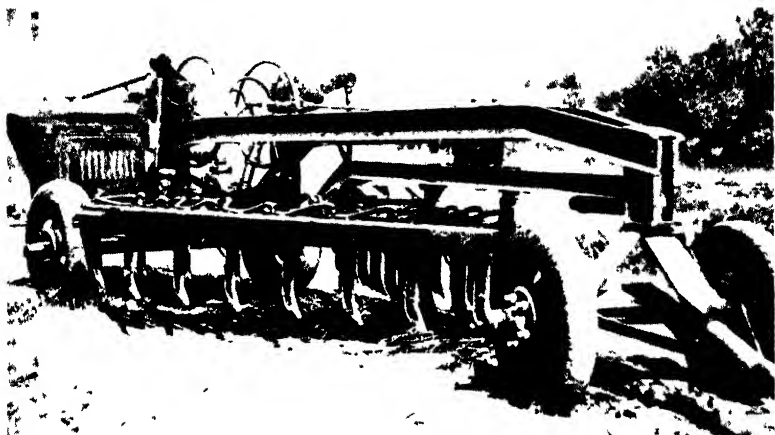


FIG. 152 —Sub-oiler for applying bituminous material in soil stabilization

described by F. V. Reagel (154). The clay surface was scarified, and the loosened material windrowed and mixed with the desired percentage of bituminous material by means of a traveling mixing plant. Varying quantities of aggregate were added prior to scarification to test the efficiency of additional friction material in case too great a degree of plasticity should be induced and also to check the relative efficiency from an economical point of view.

About 35 gal. of water per cubic yard of mixed material was added to facilitate mixing in the pug mixer. Reagel reported that the sections that had had time to dry prior to freezing carried traffic with little or no signs of distress, although there was some evidence of a sponginess or rubbery action indicating that future additional consolidation would and should take place.

Stabilization with Tar.—The use of tar in soil-road stabilization has been described by J. S. Williamson (155). The tar used consisted of two grades: Grade 1 had a specific viscosity of 4 to 8 (Engler) at 40°C.; and grade 2 tar a specific viscosity of 26 to 36 (Engler) at 50°C.

After the road was graded and shaped, the soil was pulverized for a width of 22 ft. Grade-1 tar was added to the loosened soil, thoroughly mixed in, and the mixture windrowed. Grade-2 tar was added later, and the mixture spread and compacted by traffic.

The moisture content of the soil varied from 2.9 to 7.5 per cent at the time when the tar was added. About 1.4 gal. of grade 1 tar per square yard and about 2.0 gal. of grade 2 tar per square yard were required.

A tack coat of about 0.26 gal. of grade 2 tar per square yard, covered with sand, was applied to the surface to prevent dusting under traffic. A bituminous wearing course was to be applied later.

From observations made soon after this section of stabilization was constructed, Williamson noted the following:

1. A very light-viscosity tar mixed with soil results in a mixture extremely resistant to water.
2. Unless a tar mix is closed by a seal coat very soon after it is mixed, a portion of the tar will apparently disappear.
3. Small lumps of clay soil do not seem to cause any trouble if the lumps are surrounded by material impervious to water and protected from traffic.

Problems

1. Outline the procedure followed in the construction of a stabilized-soil road by means of (a) plant mix, (b) road mix, and (c) cut-and-try method.
2. What are the principal pieces of equipment required for each type of construction mentioned in problems 1?
3. If testing equipment were not available, in what way could one judge the suitability of material for a stabilized-soil road?
4. What quantities of (a) calcium chloride and (b) sodium chloride are usually used in road stabilization?
5. In what condition should a soil be with regard to moisture content when mixed with Portland cement?
6. Should the mixed soil and cement be wetted before compaction? How much?
7. Should a soil to be treated with bituminous emulsion be wet or dry before the application of the emulsion?

8. (a) How is mixing usually accomplished on a bituminous emulsion-stabilized road?

(b) How soon after mixing can rolling proceed?

9. What other methods of stabilizing soils with bituminous material have been tried?

10. What is the purpose of a surface treatment on a stabilized-soil road?

11. Why is it that a material that serves excellently as an untreated road surface may fail when covered with a bituminous surface?

CHAPTER XV

SOFT FOUNDATION SOILS

The treatment of soft undersoils varies with the type of structure to be supported and to some extent with the type of soil. It may be desirable to float a fill or displace the undersoil, depending on the manner in which the embankment is constructed. Support for structures may be improved by pile foundations or stabilization of the undersoils with insoluble binders. Soft subgrades due to removable water or frost might be improved by drainage. Examples requiring unusual treatment are cited to supplement the more common practices to illustrate the variety of methods that may be used.

Construction of Fill by Floating.—In fill construction, foundation soils of the A8 group are the principal offenders. There are two courses of procedure—to float the fill on the soft undersoil or to displace the soft soil so that at least part of the fill material rests on firmer support deeper in the soil profile. If it is desired to float the fill, the hydraulic method of construction must be employed, and only soils of the A1, A2, A3, and the more drainable varieties of the A4 groups should be used. It is only by the hydraulic method of construction that the very shallow slopes—as low as 10 horizontal to 1 vertical—required in the floating method can be used. Porous fill material is required for several reasons: (a) It furnishes high stability throughout the fill, and the fill is not jeopardized even by total saturation of its base; and (b) when the muck underlayer is compressed by the superimposed fill, water can escape freely through the porous fill, and settlement of the muck takes place quickly.

Rapid fill settlement is desired primarily because of the necessity for constructing the pavement on the fills as soon as possible. However, the rapid settlement of fills has a deeper and more important significance in regard to fill stability.

As the muck layer consolidates, its resistance to lateral flow or sliding increases. The more quickly the consolidation of the

undersoil occurs the more rapidly does the factor of safety against failure of the fill due to sliding of the undersoil increase. Danger of fill failure due to lateral flow of the undersoil diminishes rapidly after the construction of fills of porous materials has been completed.

There is not the same assurance of increased stability with fills of relatively impermeable materials. The consolidation of the compressible layer proceeds only as the contained water is squeezed out of the voids. If the water is removed as rapidly as it is released, the compression of the muck layer will take place in a more rapid and more uniform manner than it would if the water could not escape. With the passing of time, fills comprised of porous material resting on a much less stable layer may support abrupt increases of load without danger of failure of the undersoil by sliding. In contrast, the safe load capacity of fills of impermeable materials under the same conditions may not increase with age in the same manner.

When a fill is constructed by the hydraulic method, large quantities of water are discharged with the fill material, which must drain rapidly; otherwise it will require many years to become stable enough to support construction equipment.

Displacement of Peat by Excavation.—The removal or displacement of peat when required is accomplished by one of three methods. They are:

1. Excavation down to firm foundation.
2. Displacement by water pressure.
3. Displacement by blasting.

The method used depends on the depth and character of the peat, the proximity to existing structures, and the availability of a water supply. Where the peat does not exceed 14 ft. in depth, it is excavated down to firm material and replaced with suitable fill material. Deposits over 14 ft. in thickness are displaced by means of blasting or jetting. The presence of existing structures near the scene of operation sometimes makes blasting dangerous and undesirable. Jetting, on the other hand, may be carried on in any location, provided that a large supply of water is readily available.

The excavation and backfill method is ordinarily used where the depth of peat does not exceed 14 ft. but may also be used for

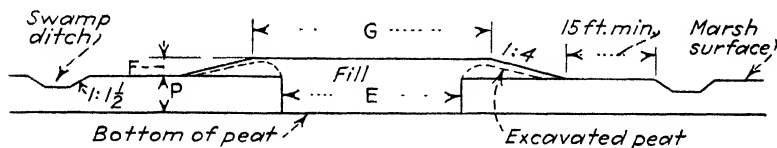
depths as great as 18 ft. if the profile discloses that in general the thickness of peat does not exceed 14 ft. An excavator with a clamshell bucket has been found the most desirable for this type of work.

Figure 153 shows a cross section of the excavation and completed fill in a dry trench for a proposed pavement 20 ft. wide. In all cases the completed fill is a minimum of 2 ft. above the marsh surface. The width of the excavation E is computed according to the formula

$$\frac{E}{2} = 12 + \frac{P}{2} \quad (74)$$

where P = depth of peat.

This is estimated to give a $\frac{1}{2}:1$ slope to the fill material underlying the pavement 20 ft. wide.



E - Width of excavation G - Width of finished grade
 F - Height of fill above marsh surface (2 ft. min.) P - Depth of peat (0 to 14 feet)

FIG. 153.—Peat excavation; dry trench.

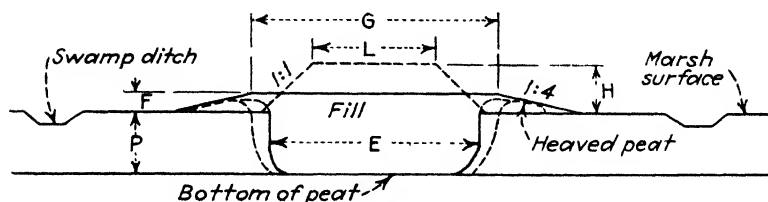
As soon as the excavation is complete, the fill, consisting of sound earth, is placed in layers of not more than 1 ft. thickness, thoroughly compacted, and to such a height and width as to conform to the plans after full compaction.

Figure 154 illustrates the excavation, loading, and completed section in a wet trench for a proposed pavement 20 ft. wide.

As the excavation is made, the backfill and surcharge, having a top width of 20 ft. and a 1:1 slope, is placed as shown to a height of 8 ft. above the surface of the marsh and maintained at this elevation by additional filling until the fill has reached a stage where the rate of settlement is not more than 0.05 ft. in 30 days, after which the surcharge is spread and thoroughly compacted to form the cross section and grade shown on the plans, the excess earth being used uniformly to widen the grade and flatten the slopes on each side.

If the fill under water is made of sand or gravel, and the fill above water is placed in layers of not more than 1 ft. in thickness and thoroughly compacted, the surcharge loading may be omitted.

The excavated peat may be used in the fill beyond the outside limits of the peat excavation or in uniformly widening the grade and flattening the slopes on each side—provided, however, that all peat placed in the upper 1 ft. of grade is so mixed with other earth as to form firm shoulders and slopes. The surplus peat is uniformly spread outside fill slopes in a manner such as not to interfere with natural drainage or swamp ditches.



E-Width of excavation

F-Height of fill above marsh surface (2 ft. min.)

G-Width of finished grade

H-Height of loading above marsh surface (8 ft. min.)

L-Top width of loading (20 ft. for proposed 20 ft. pavement)

P-Depth of peat (0-14 ft.)

FIG. 154.—Peat excavation; wet trench.

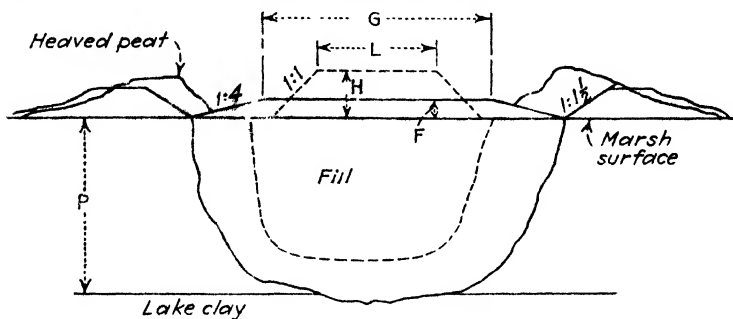
Displacement of Peat by Jetting.—The loading and completed fill section on peat to be displaced by jetting are shown in Fig. 155. The procedure for excavating, loading, and jetting is as follows:

The surface mat of peat is excavated to a depth of 4 ft. and a bottom width of 24 ft., and the first loading is applied. The fill, having a top width of 20 ft. and 1:1 slopes, is placed on the road center line to a minimum height of 8 ft. above the swamp level and maintained at this elevation by additional filling until the rate of settlement is not more than 0.1 ft. per day for 5 days.

After the preceding stage of settlement has been reached, borings are made with a light wash-boring outfit for the purpose of determining the depth and character of the fill penetration.

If the borings disclose that the fill has not settled to the bottom of the peat, the jets, consisting of 1¼-in. standard pipe with ¾ in. diameter openings in the bottom, are lowered to the bottom of

the peat by jetting. The location and number of jets used depends on the character of the penetration as shown by the borings. A typical layout of jetting apparatus is shown in Fig. 156. An 80-hp. motor, weighing about 5,000 lb. and mounted



- F*-Height of fill above marsh surface (2 ft. min.)
G-Width of finished grade
H-Height of loading above marsh surface (8 ft. min.)
L-Top width of loading (20 ft. for proposed 20 ft. pavement)
P-Depth of peat (more than 12 ft.)

FIG. 155.—Fill settlement; blasting or jetting.

on a 7-ton truck, is used for driving the pump, which is capable of developing 100 lb. pressure in the line, although the average operating pressure is 70 lb. Twenty jets of the type described above may be operated at one time.

The jetting is continued until the subsidence of the top of the fill indicates complete displacement of the peat. Before removing

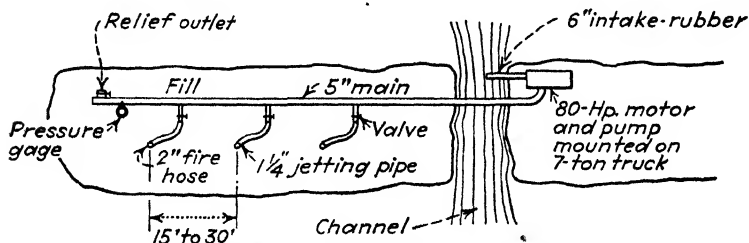


FIG. 156.—Layout of jetting apparatus.

the jetting equipment, soundings are made to ascertain if the peat is entirely displaced. Figure 157 is a cross section of a sand fill placed on a peat deposit about 30 ft. in depth and settled by jetting.

The surcharge loading, having a top width of 20 ft. and 1:1 slopes, is brought to the same elevation as the first loading and maintained to this elevation by additional filling until the fill has reached the stage where the rate of settlement is not more than 0.05 ft. in 30 days, after which the surcharge is spread and thoroughly compacted to form the cross section and grade shown on plans. The excess earth is used uniformly to widen the grade and flatten the slopes on each side.

The method of displacing peat by means of jetting appears to give just as good results as blasting and is applicable in locations where the use of explosives would be dangerous. However, when the fill material consists of clay, there is a tendency for the water

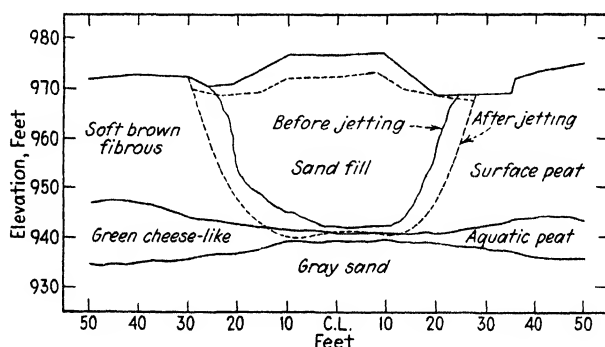


FIG. 157.—Fill settlement over peat deposit. Peat displaced by jetting. Cedar Hills State Park, T.H.M. 124, Michigan.

to rise vertically alongside of the pipe and through cracks in the fill. This causes new cracks to form and widens the old ones, through which the water escapes to the surface instead of displacing the peat as is intended. This results in a considerable increase in the expense of pumping. Also, there is a tendency for the fill material to slip out laterally beyond the roadway area. These difficulties do not arise when the fill is composed of porous materials.

Displacement of Peat by Blasting.—The method of displacing peat by blasting, as used in the state of Minnesota in peat deposits from 5 to 30 ft. deep, is as follows:

The fill material is placed over the peat to an elevation of 1 to 6 ft. above grade. Holes are bored 30 ft. apart on the center line of the roadway to solid bottom with 4-in. post-hole augers. These

holes are "sprung" at the bottom with a small amount of gelatin dynamite, then rebored and reloaded with a larger amount (50 to 150 lb.) of 40 per cent gelatin dynamite. The reloaded holes are then fired in groups of four or five.

The effect of the explosion is to lift the fill from 2 to 4 ft. and at the same time to force its underlying peat outward, thus forming a gap into which the fill material settles, resulting in a solid fill to the firm undersoil. Sand-fill material is likely to perform more satisfactorily than clay.

Both Capillarity and Permeability Considered in the Design of Drainage Systems.—The design of a drainage system that will adequately serve to remove both surface and subsurface water requires a knowledge of the following factors:

1. Character and arrangement of soil layers comprising the soil profile.
2. Character of soil moisture (capillary or gravitational) and elevation of the water table.
3. Topography of the area.
4. Climatic conditions.

The variation in the permeability of the various layers of the soil profile is the cause of and also controls the underground water movement. When the percolation rate of any one layer of soil exceeds that of the layer below, lateral movement takes place. The amount of lateral water movement, therefore, is governed by the amount of water and differences in percolation rates. Thus it is seen that water movement depends not only on the character of any particular layer but also on its relation to the other associated layers and the effect of local climatic conditions. This establishes the importance of the soil profile in the design of a subsurface drainage system.

The rate of rainfall, the time during which this rate is continued, the size of the area, and the proportion of the rainfall that reaches a given point within a given period of time are the principal factors that control the amount of water reaching a certain drain. The proportion of the rainfall that reaches a given point within a given period of time depends upon the nature of the surface, the grade of the surface, and the shape and size of the drainage area. The interrelationship of these various factors and the utilization of the existing data concerning them may best

be studied in the literature and textbooks on highway construction, airport construction, land drainage, and flood protection.

In locations where freezing temperatures occur it is necessary that the drains, in order to be effective in the time when they are most needed, be placed below the depth of freezing.

A knowledge of the coefficient of permeability serves to disclose (a) the rapidity with which precipitated water is apt to enter the topsoil and (b) the velocity with which ground water percolates through the subsoil.

A knowledge of the rapidity with which water enters the topsoil furnishes an indication of the amount of surface water to be disposed of (runoff factor), and a knowledge of the velocity of percolation furnishes an indication of the rate at which ground water is apt to reach the drains.

W. J. Schlick (21) has demonstrated that ground water is likely to flow horizontally along impervious subsoil strata toward the drains. When the drains rest above an impervious soil stratum, a portion of the water enters the drains from beneath, and the remainder passes by the drains and stays in the soil. Completely to intercept seepage, therefore, the drains must extend in depth to the impervious subsoil strata. Thus a particle of precipitated water entering the soil at the ground surface does not travel diagonally toward the drain but moves vertically downward until it both meets and joins the ground water flowing in a horizontal direction.

The foregoing discussion applies only to gravitational water or that portion of the total moisture content of the soil which moves under the influence of gravity. That portion which rises in the soil owing to surface tension, to form the "capillary fringe" above the ground-water elevation, cannot be removed by artificial drainage.

The ratio of the amount of gravitational to the amount of capillary water contained in a soil varies with change in the soil character. This is illustrated by Table 34 (21).

The thickness of the capillary fringe depends also upon the character of the soil. The top of this fringe rises and lowers with rising and lowering ground-water elevation, and as a consequence when drainage serves to lower the gravitational-water elevation (ground-water elevation) it serves to lower also the elevation of the capillary fringe.

Drainage Design Illustrated.—Figure 158 shows how drains should be placed to prevent seepage waters from entering the subgrade of a highway. In order to intercept the water before it enters the roadway, the drains must extend to the bottom of the

TABLE 34.—EQUIVALENT THICKNESS OF SOIL AND MOISTURE IN 4-FT. DEPTH OF MIXTURE
In inches

Soil type	Total thickness of soil	Total thickness of water	Thickness of capillary water not removable by drainage	Thickness of gravitational water removable by drainage
Sand	23	25	7	18
Silt loam	23	25	13	12
Clay	20	28	21	7
Muck	9	39	29	10

water-bearing stratum. Otherwise, water will enter the roadway beneath the drain and, owing to hydrostatic pressure in addition to capillarity, may be forced into the subgrade.

The method of draining a highway subgrade consisting of a layer of sand underlain by an impermeable layer having an irregular surface which causes the formation of water pockets is illustrated in Fig. 159A. Figure 159B shows an arrangement of drains to remove water from the subgrade in a location where the

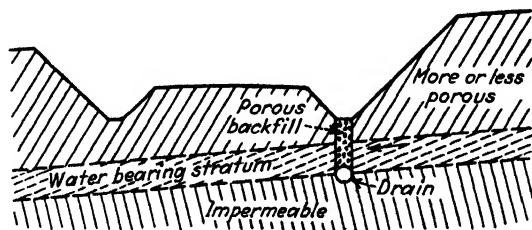


FIG. 158.—Drains should extend through the entire depth of the pervious layer and into the impervious layer.

water exists under hydrostatic pressure and the impermeable layer is at a great depth. These methods have been used successfully to prevent frost heave in more or less porous soils which do not possess capillarity in appreciable amounts.

The method of draining a frost-heaving silt possessing high capillarity is shown in Fig. 160. In this case, not only are drains

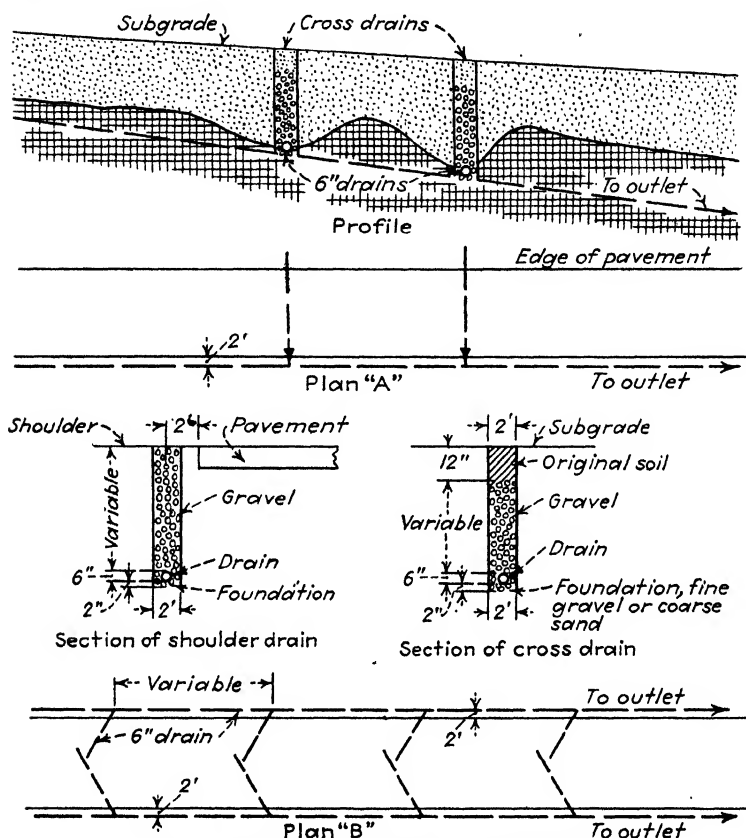


FIG. 159.—Frost-heave prevention in drainable soils.

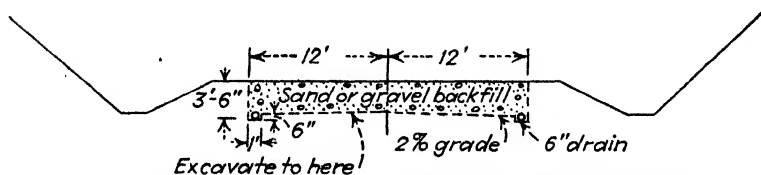


FIG. 160.—Frost-heave prevention; change of soil character by substitution.

provided, but, in addition, a definite thickness of the silt has been excavated and replaced by a porous material.

The surface of a highway is generally drained by constructing a crown in the road surface so that the water will run off into the ditches through which it is conducted away from the road. When the soils are subject to erosion, however, the water is confined within the surfaced area by means of curbs at the edges and carried from the roadway by means of troughs placed at suitable intervals. This is illustrated in Fig. 161.

Figure 162 shows various systems of laying out drains for ordinary land drainage. The spacing and depth of the drains necessarily vary with the local conditions



FIG. 161.—Pavement with curb and trough to remove surface water when soils are subject to erosion.

The foregoing designs of subsurface drainage for highways may be applied also to airport runways, and the systems of land drainage are applicable to airport drainage also.

The removal of surface water may require the installment of catch basins, examples of which are given in Fig. 163.

The design details of subsurface drains are shown in Fig. 159.

Stabilization of Undersols by Injection Methods.—The injection method consists essentially of the introduction of stabilizing materials into undersoil by means of well points or similar devices suitably spaced within the area to be consolidated.

The primary object of the injection of materials into undersols is to consolidate them (a) so as to prevent the detrimental settlement of structures already resting on them; (b) in order to permit

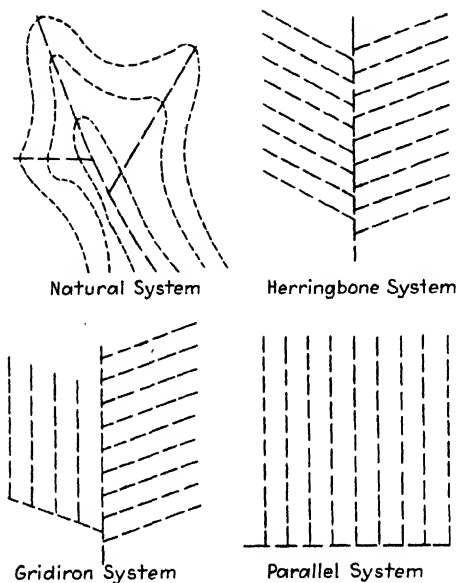


FIG. 162.—Types of subdrainage systems.

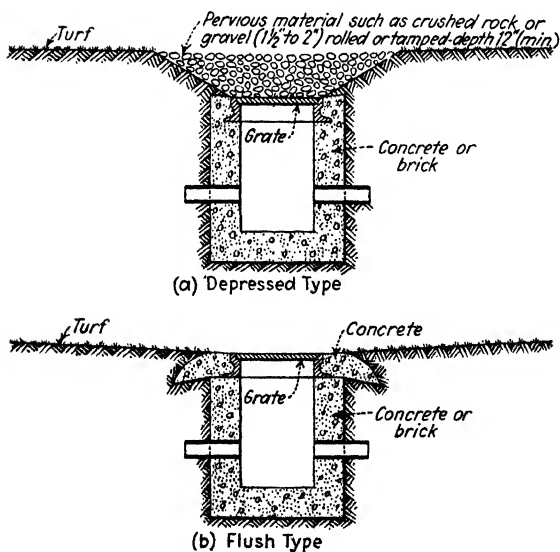


FIG. 163.—Examples of catch basins.

a more practical and economical design of foundations; and (c) as a form of underpinning in connection with excavations adjacent to old structures.

Three methods have been used for this purpose:

1. Injection of dry Portland cement into water-bearing sand and gravel (156, 157).
2. Injection of Portland-cement grout for (a) consolidating sand and gravel (158), (b) sealing seams in rock (159), and (c) stabilizing a clay fill (160).
3. Injection of sodium silicate and calcium chloride for consolidating stratified sands and gravels (161).

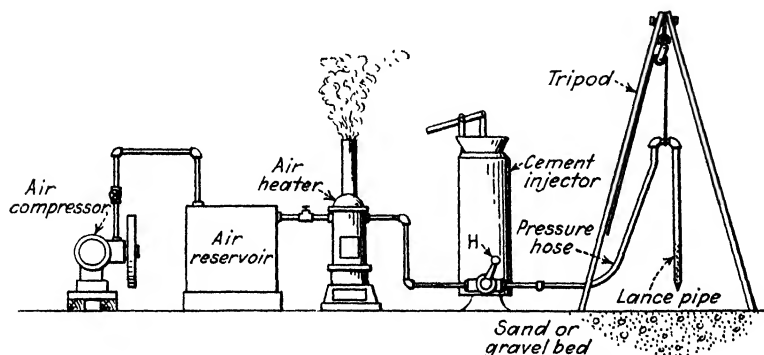


FIG. 164.—Apparatus for injection of dry cement.

The equipment consists essentially of an air compressor or a pump to force the admixture into the soil, the well points, driving and extracting rig, containers for the materials, pressure hose pipes, numerous valves, etc., depending upon the setup.

In the dry-powder process the dry cement is loaded into an injector and forced into the ground by means of heated compressed air by a mechanical arrangement such as is shown in Fig. 164. The air compressor keeps a supply of compressed air in the air reservoir, which blows a current of air through the air heater to the injector and thence through the pressure hose to the well point or lance point.

The well point is forced down into the soil to the desired depth of treatment, the penetration being facilitated by compressed air escaping from perforations in the point. This also prevents the sand from clogging the holes. The handle of the injector is then turned slowly; the air picks up the cement and carries it into

the lance pipe and out through the perforations into the sand bed. The cement particles are wetted by the ground water, while the air escapes into the atmosphere. The lance pipe is gradually raised as the bed is impregnated. Finally the compressed air is cut off, and the pipe is fully withdrawn and moved to a new location 8 to 12 in. away.

The blast of air thoroughly mixes the cement and sand, and the ground water hydrates the cement. Each time that the lance point is jetted down into a new position, the cement chamber of the injector device is loaded with a new charge of cement sufficient

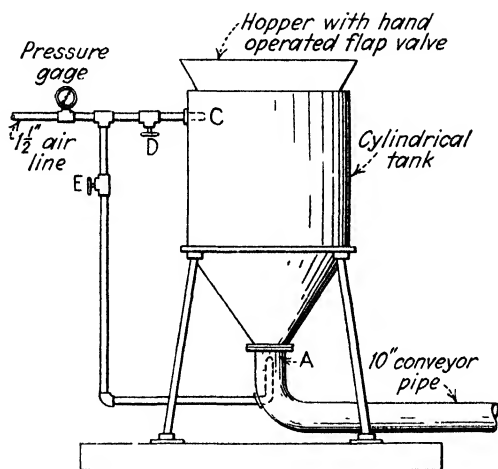


FIG. 165.—Grout machine.

to impregnate the new area. This process can be used only in granular materials such as sands and gravels.

The cement-grout process, in addition to the equipment just referred to, requires also a grout mixer and injector (see Fig. 165). After the charge is placed in the grout chamber, a lid is clamped on tightly, and the compressed-air valve opened until the pressure in the grout chamber becomes high enough to open the discharge valve. Opening of valve *E* just prior to placing the charge in the grout chamber serves to prevent the jet *A* from becoming clogged and also produces an agitation which serves to mix the grout. When it is desired to eject the grout, the valve *E* is closed, and the valve *D* opened.

As the soil pores are impregnated with the grout, the well points are gradually lifted toward the surface of the ground, as in the case of the dry-powder process. When it is desired to close seams in rock, the pipes may be sunk several feet into the bedrock, and the very wet grout forced into the seams. Air pressure of 80 to 110 lb. per square inch may be required.

When clay soils are to be injected with grout, small-diameter pipes may be driven down to the original ground level and cored out with an auger. The upper ends of the pipes are then connected to the grouting hose. In this case, pressures up to as much as 300 lb. per square inch may be required.

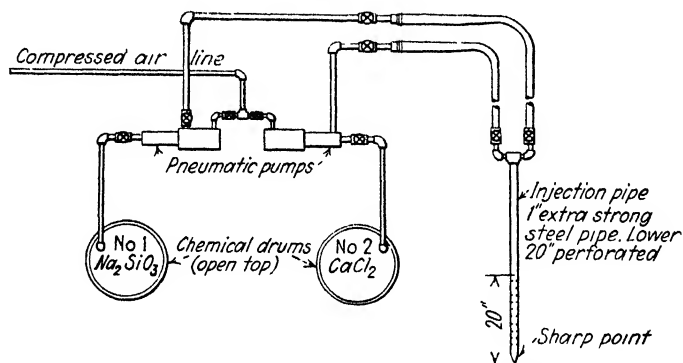


FIG. 166.—Layout for chemical-injection apparatus.

It has been observed that cement grout injected into a clay fill may spread out in layers, presumably between the compacted layers of rolled earth. The injection of the grout into sand and gravel, in contrast, has been found to result in a penetration of the grout into the pores of the deposit, forming a Portland-cement concrete.

The chemical-injection process is patented by Dr.-Ing. Joostin of Nordhausen, Germany. It differs from the cement-grout injection process chiefly in that two injections are required in each hole, as two different chemical solutions are used. Another difference is that a direct-acting force pump is used.

A layout diagram of the apparatus used for injecting sodium silicate and calcium chloride into the undersoil is shown in Fig. 166. The injection of the chemicals is accomplished by first sinking the injection pipe to the bottom of the layer to be

consolidated. Sodium silicate solution is pumped through the pipe, which is raised 20 in. (perforated length of pipe) at a time until the entire thickness of the layer is treated. The pipe is then taken out of the hole, purged of the first chemical, lowered to the bottom of the original hole, and the same process is repeated with the introduction of calcium chloride.

Pile Foundations.—Piles used in foundation work may be of two types—sheet piles and bearing piles. They may be of timber, concrete, or specially designed steel shafts. Bearing piles are those driven to support a structure, and, where possible, the length should preferably be such that the tip is penetrated slightly into a hard understratum of soil. Under these conditions, the piles are ordinarily assumed to carry the entire superimposed load, since they are more rigid than the soil between them. However, when the thickness of compressible under-soil is too great to be entirely penetrated by piling, the latter can be considered as performing the functions of reinforcement of the soil as well as a means for effecting greater distribution with a consequent lowering of stress in the compressible stratum.

Timber piles will last for centuries if permanently below the water level and protected from wood borers, such as *Limnoria* and *teredo*. When alternately exposed to air and water, however, they can be expected to decay in from 10 to 15 years, depending upon the kind of timber and the climatic conditions. The timber, according to specifications of the American Association of State Highway Officials (162), may consist of any species that will satisfactorily stand driving. The number of piles, the spacing, and position depend entirely upon the loads to be carried. However, they should not be so closely spaced that the earth between them loses its supporting capacity [see Williams (163)].

It was previously suggested that disturbing the natural structure of the soil by driving piles might cause a reduction in the bearing capacity of the soil. Because of this, the supporting capacity of a group of piles can actually be diminished by driving additional piles, as such additional piles cause the group to act as a unit instead of each pile acting separately. This is because the bearing capacity of the pile depends upon several factors: (a) the friction between the sides of the pile and the earth penetrated and (b) the bearing capacity of the soil under the tip of the pile. Therefore, a drop in the bearing capacity can be expected

when the piles are driven so close that the friction of the periphery of the group becomes the acting influence instead of the sum of the peripheries of the piles taken separately. According to Williams (163), "A minimum effective spacing is frequently stated to be about 3 ft. for piles of ordinary size, or perhaps three diameters center to center." Piles have been driven slightly closer than this in some instances, but it is probable that the same bearing could have been obtained with fewer piles. Bearing piles are usually about 20 to 30 ft. long, although in swampy and some other regions greater lengths may be required.

Specifications of the American Railway Engineering Association (163) require that for first-grade piles "the minimum diameter at the tips of round piles shall be 9 in. for lengths not exceeding 50 ft. and 7 in. for lengths over 50 ft. The minimum diameter at one-fourth length from the butt shall be 12 in., and the maximum diameter at the butt shall be 20 in."

Timber piles may be pointed where soil conditions require it, and, when necessary and under certain conditions, they may be shod with metal shoes of a design to prevent splintering when penetrating hard soil stratum and may be provided with metal collars or bands at the top to provide against splitting and brooming under the blows of the driving hammer.

Concrete piles may be either precast or cast in place. The precast piles are always reinforced. The cast-in-place piles may be constructed in several ways. A tapered sheet shell or casing, with the aid of a collapsible core, may first be driven, after which the core is withdrawn, and the casing filled with concrete. Steel reinforcing may be introduced before the placing of the concrete. In another type the steel case is gradually withdrawn as the concrete is filled into the cavity. A ram may be used to assure the complete filling of the hole. In a third type a core is used with the casing which when withdrawn leaves a hole 4 or 5 ft. below the bottom of the casing. By severely ramming the concrete as it is poured into this hole, the soil is pressed away in such a manner that the compact concrete acts as a pedestal for the shaft of the pile in the casing above.

Precast concrete piles may be of a number of shapes. If a square section is employed, the A.A.S.H.O. specifications (162) require that the corners shall be chamfered at least 1 in. Preferably, the piles should be cast with a driving point.

In the use of steel sheet piles, the principal requirement is that the sections, when assembled in place, shall be practically watertight at the joints.

Methods of Driving.—Piles may be driven by means of gravity hammers, steam hammers, or a combination of water jets and hammers. The latter should preferably be used in driving concrete piles. Gravity hammers for driving temporary piles, when permitted, should weigh not less than 2,000 lb., and the height of fall should, in no case, exceed 20 ft. Steam hammers are preferable in the driving of concrete piles. The total energy developed by the hammer should be not less than 6,000 ft.-lb. When water jets are used, the number of jets and the volume and the pressure of water in the jet nozzles should be sufficient freely to erode the material adjacent to the pile. The plant should have sufficient capacity to deliver at all times at least 100 lb. per square inch pressure at two $\frac{3}{4}$ -in. nozzles.

Determination of Bearing Values.—The size and number of bearing piles may be determined by actual loading tests or estimated by the use of formulas. When loading tests are used, the safe allowable load may be considered as 50 per cent of that load which, after a continuous application of 48 hr., produces a permanent settlement not greater than $\frac{1}{4}$ in. measured at the top of the pile. In the absence of loading tests, the safe bearing values for timber piles shall be determined by the following formulas (162):

For gravity hammers,

$$P = \frac{2WH}{S + 1.0} \quad (75)$$

For single-acting steam hammers,

$$P = \frac{2WH}{S + 0.1} \quad (76)$$

For double-acting steam hammers,

$$P = \frac{2H(W + Ap)}{S + 0.1} \quad (77)$$

where P = safe bearing power, pounds.

W = weight of striking parts of hammer, pounds.

H = height of fall, feet.

A = area of piston, square inches.

p = steam pressure, pounds per square inch at the hammer.

S = average penetration, inches per blow for last 5 to 10 blows for gravity hammers and last 10 to 20 blows for steam hammers.

The preceding formulas are applicable only when

1. The hammer has a free fall
2. The head of the pile is not broomed or crushed.
3. The penetration is reasonably quick and uniform.
4. There is no sensible bounce after the blow.

Twice the height of the bounce shall be deducted from H to determine its value in the formula.

When water jets are used in connection with the driving, the bearing power is determined by the foregoing formulas, from the results of driving after the jets have been withdrawn; or a load test may be applied.

Formula (75) is known as the *Engineering News* formula and is widely accepted because it has been observed to be closely in accord with practical results. This is especially true where the tip of the pile extends to a firm understratum of soil. Because of the fact that the pressure bulb around an individual pile does not extend to the same depth as a pressure bulb around a number of piles driven in the same location, there has been criticism of the use of this formula where the thickness of the compressible soil layer is too great to be entirely penetrated by the pile. This criticism seems well founded and should be considered by the engineer in the use of the formula.

After being driven, the tops of all timber piling are sawed off to a true plane and capped with timber or concrete in preparation for receiving the footings of the superstructure.

Examples of Unusual Problems.—The following examples¹ of foundation conditions encountered in practice illustrate the diversity of problems that must be met with unusual construction procedures.

In certain parts of the world it is not practical to use wood piles because of termites. Where it is uneconomical to resort to the more elaborate types of foundations, sand and rammed-soil

¹ From a digest of current literature prepared by C. M. Johnston, junior civil engineer, U. S. Bureau of Public Roads.

piles have been employed. The sand pile (164) is constructed by driving a large-bore pipe, closed at the lower end by a loose-fitting cap or point, into the ground and filling the hole with sand as the pipe is withdrawn. In the rammed-soil or "compressal" (165) type of foundation a hole is drilled by repeatedly dropping a long cone-shaped ram from a considerable height. Selected soil is rammed into this cavity by dropping a hammer shaped like the frustrum of a cone. Ramming is continued until the rammer does not penetrate the soil appreciably.

Quicksand is often encountered in foundation work and usually requires special treatment, particularly where near-by foundations may be seriously affected. A novel method of sinking a foundation for a boring mill at the General Electric plant at Schenectady, N. Y., was described in the *Engineering News-Record* (166). The boring mill had a shaft extending 22 ft. below floor level. It was known that a layer of quicksand occurred 11 ft. below floor level, and since precisely adjusted machines were already in place, great care was exercised in preparing the foundation for the boring mill. An open-top steel cylinder was set in place. The bottom of the cylinder was closed with a special casting which was ribbed and flanged and slightly convex. Forty-four holes in the bottom plate were fitted with gate valves and connecting pipes. Water forced through the holes washed the sand from underneath the cylinder and allowed it to settle into position. Tipping was noted but was corrected by closing the proper valves. The pipes were removed, and the bottom of the cylinder covered with concrete.

The powerhouse of the Canal and Claiborne Electric Railway at New Orleans (167) was constructed on the river front consisting mainly of made ground. At a depth of about 35 ft. this land was underlain with a natural stratum of gravel and shells varying in thickness from 3 to 7 ft. It was decided to make the foundations of piles capped with concrete, but each portion of the building and its equipment was carried on a separate cluster of piles. Piles were driven into the harder stratum with little effort. However, after standing several hours, the same blow that drove the piles 4 to 12 in. failed to drive them more than $\frac{1}{4}$ in.

An example of the use of a great number of piles for support of a building 130 ft. high has been described (168). About 18,000 piles were driven in an area 200 by 600 ft. through a very soft layer 30 ft. thick into a stratum of hard sand and gravel.

Problems

1. In constructing fills on soft foundation soils of the A8 group, what procedures may be used?
2. Why should porous material be used in the construction of fills on group A8 subgrades?
3. Discuss the difference in behavior of porous and impermeable fill materials placed on muck under soils.
4. What are the methods used for displacing peat or muck under soils?
5. What are the factors to be considered in the design of a drainage system for wet soils?
6. Soils of which group are best benefited by underdrainage?
7. Where should the drain be placed in order to intercept seepage in a water-bearing stratum located between two impermeable-soil layers?

CHAPTER XVI

PROPERTIES AND PERFORMANCE OF TYPICAL SOILS

The properties of a number of typical soils which have been used for foundation, fill, road surface, base course, and subgrade are reviewed in order to illustrate application of information furnished by soil studies in construction practice.

Muck Foundation for Fill.—Reference was made in Chap. III to the fact that portions of the Mount Vernon Memorial Highway traversed tidal muck flats adjacent to the Potomac River and were constructed by placing hydraulic fills of sand and gravel dredged from the river bottom; also, that six months after the construction of the fill at Four Mile Run, borings were made which furnished samples of the underlying muck deposit. The muck sample used for illustrative purposes in this book is typical of the deposit.

The fill was constructed in two lifts (see Fig. 18, Chap. III). The first was constructed by pumping material continuously into place to an elevation of about 5 ft. through a single pipe line. The second lift was constructed in three sections as regards time of placement and two as regards the position of the pipe discharging fill material. From the north end of the fill to approximately station 186 the material for the second lift was pumped through two pipe lines spaced about 50 ft. apart, one pipe discharging about 25 ft. to the right, and the other about 25 ft. to the left of the center line as shown in Fig. 167. Figure 168 shows a view of the fill at this stage of the construction. The two pipe lines were moved close together at the center line, as shown in the lower sketch of Fig. 167, for the construction of the remainder of the second lift.

During the construction of the second lift with the pipe lines separated 50 ft., sliding of large portions of the fill occurred at the two locations designated in Fig. 18. When the slides began, cracks formed around the area adjacent to the discharge pipe and widened as the pumping continued. The fill settled vertically,

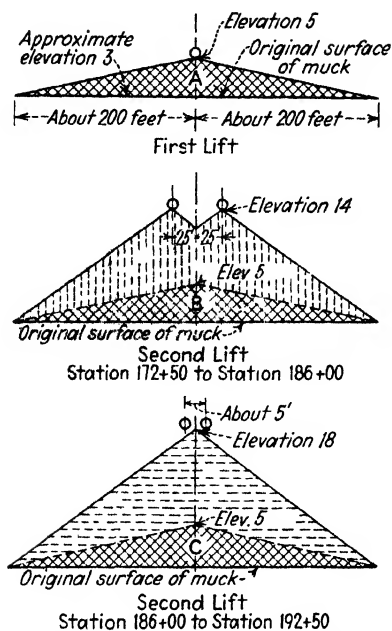


FIG. 167.—Location of pipe lines at various stages of construction.



FIG. 168.—Double pipe line for second lift.

and displacement took place laterally. With the continuation of these phenomena, mounds of soft river muck were forced up to a height of as much as 8 ft. above the original river bottom, thus

Fig. 169



FIG. 169.—Left side of fill, showing area of bulged material and nature of slide. The row of stakes in the foreground was on the shoulder line of fill before the slide occurred.

disturbing the original muck layer for distances of several hundred feet from the center line. The pumping was continued until the lateral flow stopped, and the fill reached the required height.

TABLE 35.—COMPUTED AND MEASURED MOISTURE CONTENTS

Material from core	Moisture content	
	Computed	Measured
1	123	108
2	60	76
5	123	125
6	77	79
9	86	105
10	109	116

Subsequently elevations were measured at regular time intervals in order to determine the rate at which settlement occurred.

Figure 169 shows a mound produced by the slides. A close-up of one of the mounds which shows the character of the displaced

river-bottom material after exposure to the atmosphere is shown in Fig. 170. Figure 171 shows the cross sections of the fill at four different locations as indicated by the borings.



FIG. 170 — Material displaced and shoved up by settlement of fill

From the compression-test data it was possible to compute both the moisture contents at the locations from which the core samples were taken and the fill settlements

TABLE 36 — ESTIMATED SETTLEMENTS DUE TO LATERAL FLOW WHICH OCCURRED DURING CONSTRUCTION AND OBSERVED¹ AND COMPUTED SETTLEMENTS WHICH OCCURRED AFTER CONSTRUCTION
In feet

Number of months	c l 175 + 50			c l 178 + 50			c l 184 + 00		
	Computed		Measured	Computed		Measured	Computed		Measured
	Core 2	Core 10		Core 2	Core 10		Core 2	Core 10	
1	0 3	0 9	0 4	0 2	0 9	0 2	0 2	0 4	0 3
6	0 6	2 2	0 9	0 5	1 8	0 7	0 4	1 5	0 9
12	0 8	2 8	1 1	0 6	2 1	0 9	0 6	2 2	1 5
20	0 9	3 1	1 4	0 7	2 2	1 1	0 7	2 9	1 9
36	1 0	3 3	1 6	0 8	2 3	1 3	1 0	3 7	2 9

¹ Measurements on center-line locations continued after construction of pavement

Comparison of the computed and observed moisture contents and settlements are shown in Tables 35 and 36. The observed

moisture contents were determined by drying the soil samples in the laboratory. The observed settlements were determined by levels taken periodically after fill construction.

Table 36 shows the measured settlements and also those estimated from data furnished by tests of core-2 material, representative of the stiffest, and core-10 material, representative of the softest muck in the deposit.

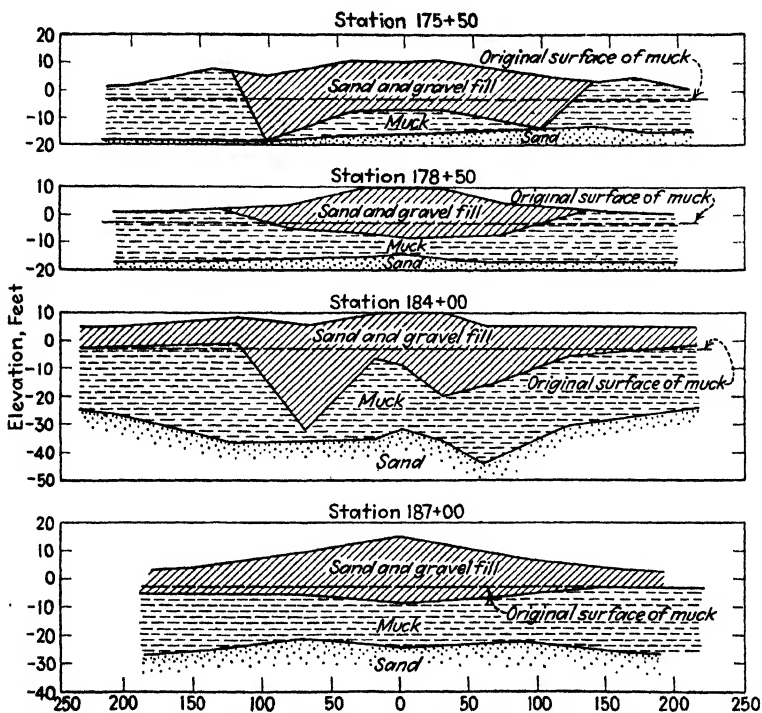


FIG. 171.—Cross sections of fill as indicated by borings.

In all cases the observed settlements are within the range of the computed maximum and minimum settlements. This indicates that if computations are based upon the compressible properties of the softest soil in deposits, the errors in the computed settlements will be on the side of safety.

Clay Soil Used as Foundation for Building.—The results of compression tests performed on a sample of clay soil extracted from a depth of 39 ft. below the ground surface at the site of a

proposed building are shown in Figs. 172 and 173. Figure 174 shows the soil profile at this location. Figure 172 shows the pore ratio-pressure curves for the sample tested in both the undisturbed and the remolded condition. The time-consolidation curve and the estimated final and periodic settlements produced in the clay-soil layer under a load of 4 tons per square foot are given in Fig. 173.

The pore ratio-pressure curve for the undisturbed material discloses that the clay soil, which has a natural moisture content of 17 per cent, exists in a very highly consolidated state. In fact, the curve resembles that of a fine, cohesionless sand rather than that of a clay soil. This is highly significant with respect to the behavior of this stratum of clay soil under building loads. Settlements occurring in sand layers supporting buildings are usually limited in amount and take place during the construction period. The high supporting value of this clay soil is further indicated (Fig. 173) by the estimated settlements under a load of 4 tons per square foot. The total settlement to be expected is 4.5 in. with 83 per cent of this, or 3.7 in., occurring within the first year.

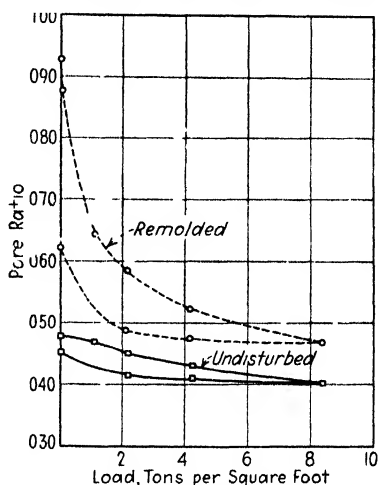


FIG. 172.—Pore ratio-pressure curves for foundation soils.

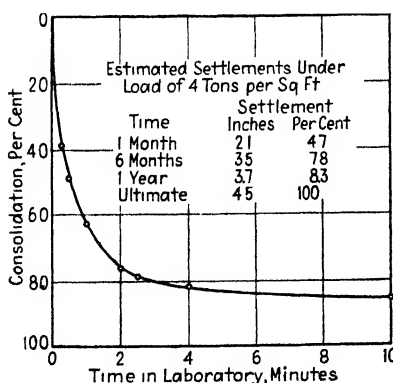


FIG. 173.—Time-consolidation curve for foundation soil.

The peculiar nature of this clay soil is indicated by a comparison of its physical constants as disclosed by laboratory tests with those of an average soil containing the same amount (52 per cent)

of clay. An average soil containing 52 per cent of clay would be expected to have a liquid limit of 72, a plasticity index of 36, and a shrinkage limit of 12. The clay soil, however, has a liquid limit of 31, a plasticity index of 15, and a shrinkage limit of 17. It is of

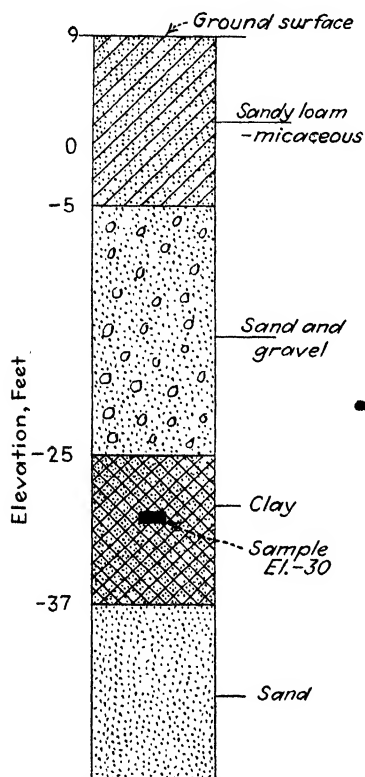


FIG. 174.—Profile of foundation soil.

interest to note also that the natural moisture content of the soil—17 per cent—is not significantly greater than the plastic limit of 16 per cent. Since even with manipulation a soil does not lose its stability until mixed with moisture somewhat exceeding the plastic limit, the effect of the high degree of consolidation for insuring stability becomes evident.

The initial pore ratio of the undisturbed sample was 0.477. After mixing the soil with water to a moisture content above the liquid limit, a load of about $7\frac{1}{2}$ tons per square foot was required to reduce the sample to a pore ratio of 0.477, the pore ratio of the undisturbed consolidated soil. Although this is not assumed to be an accurate measure of pressure under which the soil layer was consolidated, it does give an idea of the pressure required to compress a newly

laid deposit of this material to its present condition.

Assuming an average weight of 100 lb. per cubic foot for the soil above the sample, and neglecting hydrostatic uplift, the present load on the sample equals about 3,900 lb. per square foot. Under this load, according to the remolded curve in Fig. 172, the clay soil would have been compressed only to a pore ratio of 0.595, which is appreciably higher than the present pore ratio of 0.477. This indicates that the clay soil was at one time consolidated under a much greater load than that due to the present overburden

and that it has maintained this high degree of consolidation even after the removal of much of the original overburden. This may be explained by the impermeable character of the clay soil which prevented the absorption of water and the expansion that might result therefrom.

In support of the preceding test data, a geological examination disclosed that the clay soil belongs to the Raritan formation which was deposited many millions of years ago. The estimated thickness of the original overburden deposited on this formation in the location under consideration amounts to about 400 ft. or more, but much of this has since been eroded away.



FIG. 175 —Stable fill composed of selected graded material.

Soil Used in Fill. Compaction Method.—In Chap. X attention was called to the fact that the height of the fill is an important consideration when cohesive materials are used. Thus the risk of failure is very great when clay soils that have proved satisfactory in the construction of relatively low fills—say, heights up to 8 or 10 ft.—are used in fills of 15 or 20 ft. or more in height. This is illustrated by the fills shown in Figs. 175 and 176, which were constructed in the District of Columbia.

In the fill (Fig. 175) with a maximum height of about 30 ft., specially selected graded material was used and consolidated by the compaction method. This fill as shown is in excellent condition.

Adjoining this fill, material that had proved satisfactory in the construction of low fills was employed, and the same type of

consolidation was used. As soon as this fill had reached considerable height—possibly 15 ft. or more—failure began to occur owing to sliding of the material (Fig. 176). This sliding has continued to such an extent that additional right of way has had to be purchased. It is quite possible that the trouble will not be corrected until some of the cohesive material in the bottom of the fill is removed.

Table 37 shows the grading and physical constants indicative of the performance of soils used in such fills. Sample 1 is typical of material used in the fill that has performed satisfactorily. Sample 2 is typical of materials that are productive of failure when used in high fills.



FIG. 176 — Failure of fill due to sliding of cohesive material.

Soils Used in Fill Construction. Hydraulic Method.—Borings made to locate a porous material for use in the hydraulic fill at Four Mile Run, previously referred to in this chapter, disclosed a supply of sand and gravel within the range of the dredging equipment. This material was graded from fine sand to coarse gravel and had an average loose weight of 101 lb. per cubic foot and a rodded weight of 110 lb. per cubic foot. Mechanical gradings of the fraction passing the No. 10 sieve as determined on two typical samples are as follows: (a) coarse sand, 38 per cent; fine sand, 46 per cent; silt, 8 per cent; clay, 8 per cent; (b) coarse sand, 82 per cent; fine sand, 10 per cent; silt, 6 per cent; clay, 2 per cent. The centrifuge moisture equivalents are 10 and 6, respectively.

As was previously pointed out in Chap. XIII, the use of granular materials of this type is desirable in the construction of fills to be placed on soft, unstable muck undersoils. In the construction of embankments to be placed on stable undersoils, however, such as in the case of hydraulic-fill dams, the range of suitable materials may be considerably greater.

TABLE 37.—MECHANICAL ANALYSES AND PHYSICAL CONSTANTS OF (1) SATISFACTORY AND (2) UNSATISFACTORY FILL MATERIALS
Mechanical Analysis

Soil number	Gravel	Coarse sand	Fine sand	Silt	Clay	Colloids
1	22	52	16	11	21	13
2	0	4	9	19	68	51

Physical Constants of Material Passing No. 40 Sieve

Soil number	Liquid limit	Plasticity index	Shrinkage limit	Shrinkage ratio	Moisture equivalents	
					Centrifuge	Field
1	24	6	16	1.9	22	20
2	65	47	10	2.0	31	25

Joel D. Justin (169) suggests that the suitability of materials for use in the construction of dams by the hydraulic method may be disclosed by a simple modification of the flocculation test. This modified method is as follows:

A small amount of the dried material is powdered between the fingers, dropped into a test tube, and settled by tapping. A sticker is placed on the glass tube to mark the top of the powdered-soil deposit. Water is then added and thoroughly mixed with the soil by shaking the test tube with the thumb held over its mouth. The mixture is then allowed to rest for 24 hr.

Volumes of the resulting wet sediments which exceed the volumes of the corresponding powdered samples by more than 50 per cent indicate soils that are not suitable for use in dams constructed by the hydraulic method.

In the case of "made land" built up by the hydraulic method in connection with park development, landscaping, and the like, where the degree of stability necessary for dams and highway

fills is not required, much poorer soils than those referred to above have been used successfully. In this type of development, time is not an important consideration. An example of such a development is Columbia Island in the District of Columbia. This island, on which a section of the Mount Vernon Memorial Highway has been constructed, is a body of land built up from the waste material resulting from the hydraulic dredging operations performed in deepening the Potomac River channel.

The test constants of this material are as follows: liquid limit, 56; plasticity index, 25; shrinkage limit, 34; centrifuge moisture equivalent, 64 (waterlogged); field moisture equivalent, 44.

These test constants clearly indicate the character of this material. In addition to the waterlogging in the centrifuge test, the relatively low field moisture equivalent of 44 combined with a high shrinkage limit of 34 suggests the presence of organic matter in the colloidal state. Also, the high plasticity combined with the relatively high shrinkage limit suggests the presence of diatoms. It is evident, therefore, that the material is a muck soil consisting primarily of inorganic silt, with some clay, colloidal organic matter, and diatoms.

This material is also typical of that which was used in building up the Potomac Park area in Washington, D. C., for which purpose it has been entirely satisfactory. Such material, however, should never be used in landing fields, earth dams, highway fills, and similar structures in which stability and time of completion are important considerations.

Clay Soil Used in Earth-dam Construction. Densification Method.—Results of the routine subgrade-soil tests performed on a representative sample of the material used in the Back Creek earth dam (170) (see Fig. 177) constructed by the U. S. Forest Service near Waynesboro, Va., are as follows:

Mechanical analysis, millimeters					Constants of fraction smaller than 0.42 mm.				
Particles smaller than					Liquid limit	Plasticity index	Shrinkage		Field moisture equivalent
2.0	0.42	0.05	0.005	0.001			Limit	Ratio	
76	71	46	21	11	30	12	19	1.7	24

According to these results, this typical soil may be considered to have enough sand to produce stability, enough clay to bind the sand in a watertight mass, and not enough silt to cause detrimental capillary rise. The low liquid limit for a soil of this grading indicates the absence of mica, diatoms, or organic matter. The plasticity index is high enough to indicate that the soil will not lose stability in the presence of water without manipulation. The shrinkage limit is well below the liquid limit, a condition usually met in the more stable soils. The field moisture equivalent is also well below the liquid limit, a further evidence of stability, and the optimum moisture content of the soil is very nearly equal to its plastic limit.

Field Control.—Control during construction consisted of determining the moisture content and degree of densification of

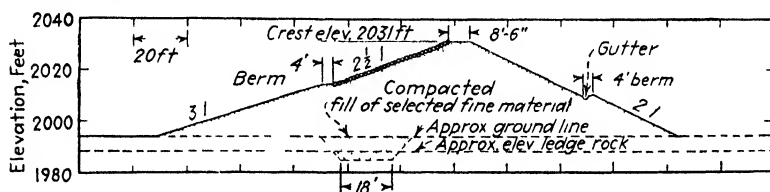


FIG. 177.—Cross section at center line of dam. Back Creek Dam near Waynesboro, Va., George Washington National Forest.

the material being placed. The moisture contents were determined by packing the wet soil in the Proctor cylinder, weighing, measuring the resistance to penetration, and comparing these values with the laboratory curves for wet weight and resistance to penetration for the particular sample. Checks on these values were made by oven drying small samples to determine the moisture contents more accurately. If the soils were found to be below the optimum moisture content, they were sprinkled with water; and if found to be above the optimum, they were spread in thinner layers and allowed to dry either before rolling or during longer rolling periods.

The extent of consolidation was determined by measuring the resistance to penetration of the soil layer in place (Fig. 178) and comparing the values with the readings obtained when the same soil was compacted in the standard manner in the cylinder. If the plasticity-needle readings of the soil layer were less than those for the soil in the cylinder, more rolling was required. This

test was not used so extensively as the dry-weight test because the presence of gravel in the material interfered with the needle.

Check tests on the degree of consolidation were made in the following manner: Approximately 15 lb. of the compacted soil was removed from the layer by a post-hole auger and weighed on a scale of 30 lb. capacity. The same soil was then packed in the Proctor cylinder, weighed, and the plasticity and moisture content computed as in the regular tests. The hole was filled with sand of a known loose weight per cubic foot, and the volume computed.



FIG. 178.—Use of Proctor apparatus for control purposes in the field.

The wet and dry densities of the compacted material were determined from the weight, moisture content, and volume. A comparison of the densities of the wet soil in place and the same soil compacted in the cylinder in the standard manner indicated whether more rolling was required or the number of trips could be decreased.

Construction Procedure.—During construction, the stream flow was taken care of by a 36-in. metal pipe encased by a 6-in. thickness of concrete supported on ledge rock.

Water was controlled by a floodgate which was mounted on a tower at the upstream end of the pipe. This tower was also founded on rock.

To prevent the creeping of water along the drainpipe, several collars were added, and very carefully selected material was hand-tamped around the pipe and the collars.

Water was carried to the drainpipe by a trench, which intercepted the stream about 500 ft. in front of the dam site. The stream was diverted into this trench by a cofferdam which consisted of truckloads of material dumped across the stream bed.

After the stream flow had been diverted through the drainpipe, and the dam site dried up considerably, the excavation of the cut-off trench was begun. The east side of the trench was dug by hand, and the dirt removed by a scraper and reworked on the back of the fill. The rock foundation for the core wall was carefully cleaned and dried—in some cases by compressed air—before any material was placed.

The west side was excavated with a shovel, and the rock cleaned off by hand. When this was finished, a number of mud seams were noticed, and some of the rock was removed in an attempt to reach a more solid foundation. As no reduction in the number or size of the mud seams was noticed after 2 ft. of rock had been removed, the excavation was discontinued, but, in order to eliminate the possible washing of the core-wall material in contact with these seams, they were capped with concrete.

After the trench had been completely cleaned and dried, the placement of material was begun. The borrow-pit area was first cleared of all stumps, and the topsoil carefully stripped off. The soil was tested for moisture content by means of the plasticity needle by the procedure outlined above and then loaded into trucks by the gas shovel and hauled to the dam. The efficiency of the shovel was greatly increased by the assistance of the trail builder in pushing the soil into the location of the shovel and so deepening the cut.

The material was spread with a bulldozer in 8-in. layers. The soil was compacted by means of two sheep's-foot rollers drawn by tractors. One of the tractors was equipped with a bulldozer and was used to spread the layers. Generally 8 to 10 trips of the roller were required to produce the desired compaction.

Lime Rocks.—Limestones include those rocks which are composed principally of calcium carbonate with varying amounts of other components, chiefly silica, magnesia, alumina, and iron

oxide. The physical characteristics vary from the crystalline marbles and consolidated true limestones to the unconsolidated shell marls. The less consolidated grades of limestone, known locally as "lime rocks," because of their availability are used extensively in the construction of road surfaces and base courses in the southeastern states.

The lime rocks are chiefly of marine origin (171). Lime, CaO , is widely distributed throughout the earth's crust. Because of its chemical activity it is always found in combination, chiefly as calcium carbonate, CaCO_3 , in the form of limestone, calcite, and marine and fresh-water shells. It is not readily soluble in pure water but is soluble in water charged with carbon dioxide, CO_2 , *i.e.*, carbonic acid, H_2CO_3 .

Rain water becomes carbonic acid by absorption of carbon dioxide from the air or by contact with decaying vegetable matter. This acidulated water percolating through soil and rock materials takes calcium carbonate into solution as calcium bicarbonate, $\text{Ca}(\text{HCO}_3)_2$. Still in solution the lime is carried by streams and rivers to the ocean where conditions are favorable for deposition.

The deposition of calcium carbonate to form limestone is effected through two main processes: (a) the activity of organisms which remove calcium carbonate from solution to form shells; (b) the removal of carbon dioxide by chemical and physical agencies, such as evaporation, aeration, and activity of bacteria and algae, with resultant supersaturation with and precipitation of calcium carbonate.

In an ideal case with both these processes functioning the sea floor is covered with more or less worn and broken shells and the remains of lime-secreting sea plants, bound together by an amorphous mass of calcium carbonate precipitated by the different chemical reactions. If the bed is buried under newer sediments owing to sinking of the sea floor, the pressure causes further consolidation and partial crystallization. If it is exposed by rising of the sea floor, rain water will leach out the calcium carbonate from the upper layers and redeposit and partially crystallize it in the lower, thus effecting further consolidation. The degree of consolidation has much to do with classifying limestones, which differ widely in composition, hardness, texture, and color. The one property common to all is the predominance of calcium carbonate.

The lime rocks are found in greatest abundance throughout the state of Florida and in the southern portions of Georgia and Alabama. Typical of the Florida deposits are the Ocala, the Miami oolite, and coquina formations. The physical characteristics and general locations of these deposits are as follows:

1. *Ocala*.—The Ocala formation consists of a cream-white soft, porous, and granular limestone which bleaches to a chalk white on exposure (see Fig. 179). Some portions are hard and semicrystalline, and in some localities it contains nodules and layers of chert. On the whole, it is extremely uniform in texture and composition, containing as high as 99.6 per cent calcium carbonate.

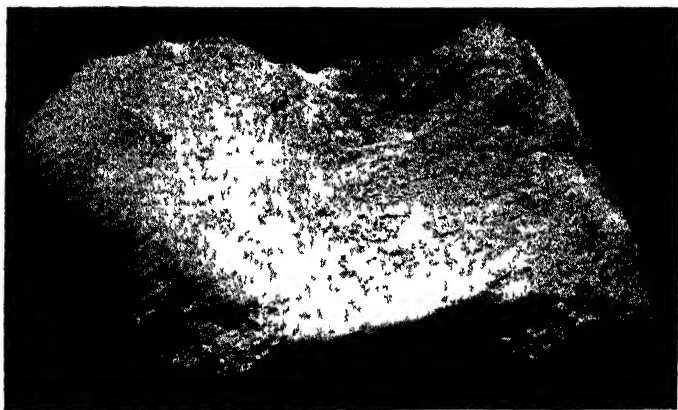


FIG. 179 —Ocala lime rock. NOTE Left half hard and semicrystalline, right half soft and porous

Although the Ocala limestone presumably underlies the whole state to a thickness of more than 400 ft., it is exposed over a comparatively limited area of about 50 by 150 miles in extent in the northwest portion of the peninsula, adjacent to and paralleling the Gulf of Mexico, and in a small area in west Florida just below the Alabama line.

2. *Miami Oolite*.—The typical rock of this formation is a soft, white, pure (95 per cent calcium carbonate) oolitic limestone with occasional irregular layers of calcite. In places, rounded grains of white sand are intermixed, sometimes occurring in lenses or pockets. The rock hardens on exposure to air and rain (see Fig. 180).

The Miami oolite (also known locally as "Ojus" rock) extends in a narrow strip along the coast of southeastern Florida.

3. *Coquina*.—The coquina limestone occurs in three principal phases. The first is composed entirely of small, clean shells and shell fragments, usually unconsolidated but with hard ledges (near the top of the deposit) of firmly cemented shells (see Fig 181) The second consists of finely crushed shells closely cemented into a hard, compact rock. The third phase contains finely crushed shells and a large quantity of quartz-sand grains, cemented together by the calcium carbonate of the shells. In the latter phase it is more a calcareous sandstone than a sandy coquina.

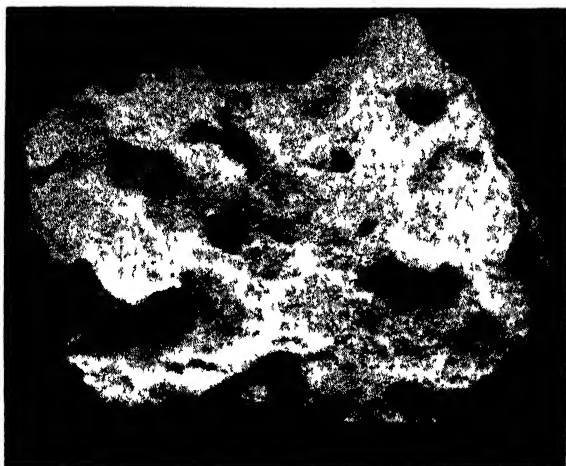


FIG 180 — Miami oolite

The coquina deposits are located chiefly in a narrow strip along the east coast of the peninsula.

The various degrees of quality or performance of the materials investigated can be designated by the terms "excellent," "good," "fair," and "poor."

The tests of greatest value for predetermining the quality of road-building lime rocks are the plasticity tests, the field moisture equivalent, and the flocculation test. A plasticity index of zero to the exclusion of all other determinations appears sufficient to indicate those lime rocks which are likely to perform satisfactorily as base-course material under practically all conditions; a plasticity index from 1 to about 7, combined with a field

moisture equivalent not exceeding about 20, lime rocks that will perform satisfactorily in carefully constructed base courses under average conditions; a plasticity index from 1 to about 7, with a field moisture equivalent greater than 20, lime rocks that will perform satisfactorily in carefully constructed base courses under fairly dry conditions; and a plasticity index of 8 or more, lime rocks that are likely to prove troublesome. The quality of material within a given deposit or formation may vary with the calcium carbonate content. Table 38 shows the results of tests indicative of the various grades of lime rocks.

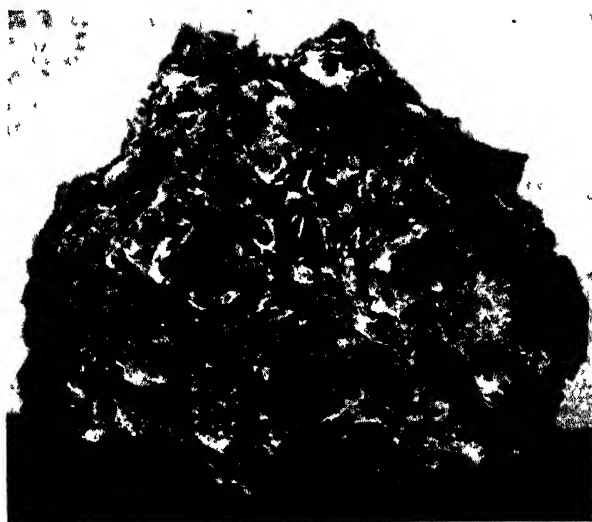


FIG 181 —Coquina (shell phase).

Caliches.—The term “caliche,” as used in the United States, is applied to a group of formations consisting of calcium carbonate and silica with smaller percentages of alumina, iron, and magnesium carbonate. These formations occur locally, in relatively thin beds or powdered form, but usually consist of clays and sands more or less thoroughly cemented by calcium carbonate, or of gravels or breccia so cemented, into caliche conglomerates.

They have been found and developed as road material (172) in extensive areas of the arid regions of the Southwest, notably in Arizona, New Mexico, northwestern Oklahoma, and western and southern Texas.

They are frequently described as being both calcareous and siliceous. The latter term arises apparently from chemical analyses which do not distinguish the cementing material from

TABLE 38.—TEST CONSTANTS OF LIME ROCKS

Sample number	Liquid limit	Plasticity index	Flocculation factor	Calcium carbonate, per cent
Excellent				
1	20	0	1 2	98 66
2	20	0	1 2	98 66
3	18	0		98 04
4	18	0	1 2	98 13
5	17	0	1 0	97 85
Good				
6	23	0	1 2	90 89
7	21	6	1 3	87 50
8	20	5	1 5	88 75
9	19	6	1 4	89 55
10	18	4	1 3	91 78
Fair				
11	26	5	1 9	95 34
12	25	5	1 7	98 75
13	24	5	1 6	95 63
14	24	5	1 3	97 14
15	24	3	1 9	98 66
Poor				
16	34	17	2 3	88 32
17	30	15	2 1	88 04
18	28	10	1 9	90 45
19	25	9	1 8	91 16
20	25	8	2 0	90 45

the chemical content of siliceous particles of clays or sands that form the base for cementation. However, one occurrence in Texas is reported definitely as having silica alone as cementing material.

The caliches of the United States of America are not related, unless in general appearance, to the nitrate deposits of Chile, to which the term caliche was originally applied.

Geologically, these caliches are superficial formations, varying from beds of incoherent powder to those of very considerable hardness and a corresponding high degree of cementation. They have limited continuity and are local in extent but are of widespread occurrence. They vary in thickness from a few inches to 60 ft. or more. Occasionally they are surface deposits, exposed by erosion (wind or water), or, quite rarely, are products of evaporation of standing bodies of water (173).

Caliches used in highway construction may be classified into three general types, according to hardness:

1. *Flourlike Caliches*.—This type consists of a fine sand loosely cemented together with a fine, impalpable powder which, according to chemical analyses, is composed principally of silica or calcium carbonate. This type of caliche may be handled or loaded in the pit without plowing or mechanical breaking of any kind. It may be scraped to a loading platform; loaded into wagons or trucks with an elevating grader, or loaded into wagons or trucks by laborers with shovels without previous mechanical manipulation.

2. *Semihard Caliche*.—This type of caliche consists of cemented areas interspersed with the flourlike caliche. Caliche beds of this nature have to be broken up with a hard-steel plow or hard-steel rooter before the material can be loaded into trucks or wagons. This type of formation can be broken up and loaded with steam shovels as well. Usually, however, when placed in the subgrade of a road under construction, the large and semihard lumps have to be broken up with mauls or hammers.

3. *Hard Caliche*.—This type of caliche consists of well-cemented strata or conglomerate areas which have to be blasted with powder or dynamite and by this means broken into lumps of variable size before the material can be hauled from its location in the pit. Usually this type of caliche must be run through a crusher before it can be used.

Plasticity indexes of less than 10 supplemented by flocculation factors up to and including 1.7 seem indicative of caliches that should serve satisfactorily under conditions of average moisture; and plasticity indexes of 10 to 15, with flocculation factors of 2.5 or

less, materials likely to be suitable only under the dryer and arid conditions. Plasticity indexes greater than 15 indicate poor-grade materials. Test data on typical samples of caliches of these groups are shown in Table 39.

TABLE 39.—RESULTS OF TESTS ON CALICHES
Group 1

Sample number	Liquid limit, per cent	Plasticity index, per cent	Flocculation factor
1	20	3	1.7
2	21	5	1.6
3	25	6	1.2
4	26	6	1.6
5	26	8	1.5

Group 2

6	30	8	1.7
7	34	10	1.7
8	31	11	1.6
9	35	12	2.5
10	38	13	2.0

Group 3

11	33	17	7.5
12	46	28	3.3
13	40	18	3.0
14	46	26	4.7
15	52	29	4.0

Graded Soil Mixtures.—Only a small percentage of the soils even in Georgia are of the highest quality, and to find natural soils of the hard, or class A, quality outside Georgia is more the exception than the rule. Of 29 federal-aid projects reported by Strahan (25), but 3 had strong, hard roadbeds, free from ruts, holes, or corrugations and indicative of the class A materials.

Generally, the admixtures and chemical treatments serve to provide the medium- and the poor-grade materials with (a) the superior consistency and binding stability acquired by the excellent class A topsoils from long periods of cultivation and weathering on one hand and (b) the stability and the hardness of the

gravel-type surface, without the dust-producing mulch, on the other.

Test data on a number of soil mixtures expressed in terms of the grading and the plasticity referred to above are shown in Table 40.

Number 1 shows the requirements of good mixtures as suggested in Chap. XI. Number 2 represents the requirements of good soil mortars based upon Strahan's work combined with coarse aggregate to an assumed amount of 50 per cent. Number 3 is typical of the mixtures used in the construction of stabilized roads in Washtenaw County, Michigan, in 1933.

Investigations by Collings and Stewart (174, 175) of the effect of calcium chloride and also soil mixture and character upon stability under controlled truck traffic on experimental sections furnished the data on five types of soil mortar (mixtures 4 to 8) and five types of sand-clay gravel (mixtures 9 to 13).

Investigations by Travers and Hicks (176) of stabilized roads in Onondaga County, New York, furnished information on mixtures 14 to 17. Additional investigations by Collings and Stewart (177) furnished data on the remaining 22 mixtures.

The sand-clay gravel mixtures represented by samples 9 to 13, inclusive, were all considerably more stable than soil mortars represented by samples 4 to 8, inclusive. This is in line with Strahan's findings: "When coarse material is added to a good soil mortar in appreciable amount (10 per cent or more), the hardness and durability of the surface is increased and continues to increase until a full gravel-type surface is reached" (24).

Of these 10 mixtures first investigated by Collings and Stewart, only those represented by 4, 5, 6, 9, and 10, with some plasticity, could be considered satisfactory. Samples 14 to 17, inclusive, representative of satisfactory mixtures reported by Travers and Hicks (176), had somewhat more plasticity than the five satisfactory soils from Collings and Stewart's experiments (177).

Of the mixtures represented by samples 18 to 39, inclusive, those represented by samples 23 to 24, with no plasticity, and by 32, 33, 38, and 39 were the less resistant.

Table 41 furnishes additional information on the grading and plasticity constants of road-soil mixtures.

Number 1 shows the requirements of good mixtures as suggested above. Numbers 2, 3, and 4 represent, respectively, the high,

TABLE 40.—PLASTICITY AND GRADING OF TYPICAL SOIL-ROAD MATERIALS
In per cent

Number	Liquid limit	Plasticity index	Passing sieves					
			1 in.	¾ in.	No. 4	No. 10	No. 40	No. 270
1	14 to 35	1 to 15	100	85 to 100	55 to 85	40 to 65	25 to 50	10 to 25
2	14 to 35	4 to 8	100	50	26 to 34	8 to 15
3	20	6	73	50	38	17
4	16	1	100	100	75	16
5	15	4	100	100	77	27
6	15	1	100	100	73	22
7	16	0	100	100	77	13
8	17	0	100	100	76	7
9	17	4	100	100	72	39	25	12
10	15	3	100	100	62	40	29	16
11	13	0	100	97	53	39	22	11
12	13	1	100	100	78	56	33	13
13	12	0	100	100	69	43	20	10
14	..	8	100	98	63	51	31	14
15	..	7	100	98	73	63	39	23
16	..	5	100	97	52	39	24	17
17	..	7	100	98	65	53	28	10
18	27	14	100	82	57	50	41	20
19	22	11	100	82	58	51	43	20
20	21	10	100	80	56	48	38	14
21	17	5	100	79	54	46	36	11
22	18	5	100	79	54	46	34	10
23	..	0	100	82	57	50	38	10
24	..	0	100	79	54	45	34	6
25	28	15	100	95	68	54	46	24
26	22	11	100	100	68	53	45	23
27	21	10	100	100	77	59	48	22
28	27	15	100	100	71	53	43	28
29	22	11	100	94	64	47	39	16
30	17	6	100	94	64	46	36	14
31	27	15	100	100	70	50	38	18
32	20	10	100	100	71	52	40	15
33	18	6	100	94	65	47	56	10
34	21	10	100	96	77	65	53	26
35	27	15	100	96	77	66	58	36
36	21	10	100	94	65	49	40	19
37	27	15	100	94	63	48	41	24
38	20	10	100	92	57	39	28	14
39	28	15	100	92	52	33	28	17

average, and low values furnished by tests and analyses of samples representative of the soils that serve excellently under all weather conditions as road surfaces in Onandaga County, New York.

TABLE 41.—PLASTICITY AND GRADING OF ADDITIONAL TYPICAL SOIL-ROAD MATERIALS
In per cent

Number	Liquid limit	Plasticity index	Passing sieves					Ratio of binder to fines
			$\frac{3}{4}$ in	No 4	No 10	No 40	No 270	
1	14 to 35	1 to 15	85 to 100	55 to 85	40 to 65	25 to 50	10 to 25	67
2	27	9	94	69	52	34	18	75
3	23	7	84	56	41	25	13	56
4	17	4	65	34	24	14	8	38
5	29	11	87	68	55	33	26	71
6	24	8	81	52	41	27	16	61
7	18	4	70	36	26	18	12	37
8		6	89	71	62	52	25	48
9		3	86	69	62	47	20	40
10		0	83	67	61	42	14	33
11		8	100	73	63	39	23	71
12		6	99	64	44	29	15	52
13		3	97	52	39	24	10	36
14	28	15	100	77	66	58	36	65
15	23	11	92	65	52	42	20	48
16	17	5	79	54	46	34	10	29
17	25	12	97	82	73	53	28	67
18	19	8	93	73	61	44	21	50
19	16	4	89	67	40	27	14	36
20	19	7			58	38	18	47
21	18	2	91	63	52	38		
22	58	39	100	99	98	90		
23	22	5	97	66	59	46	20	44

Numbers 5, 6, and 7 represent the high, average, and low values for materials that were good under all weather conditions and excellent under dry weather conditions in the same location. Numbers 8, 9, and 10 also represent high, average, and low values for materials from Onandaga County, New York. In this case they were good only under dry weather conditions.

Numbers 11, 12, and 13 represent the high, average, and low values, respectively, for six good road mixtures, two from Michigan and four from New York State.

Numbers 14, 15, and 16 are, respectively, the high, average, and low values for mixtures indicated as good by Collings and Stewart's experiments.

Numbers 17, 18, and 19 represent, respectively, high, average, and low values of good mixtures from road surfaces in Indiana.

Number 20 is typical of the mixtures used in Michigan roads as reported by J. W. Kushing (178). Number 21 is representative



FIG. 182 —View of road in Adams County, Pennsylvania, showing junction of untreated side road with stabilized surface

of the gravel, and No. 22 of the binder soil used to make up the mixtures in Indiana represented by Nos. 17, 18, and 19.

In designing the mixtures it should be kept in mind that if this type of surface is to be kept within the low-cost field, the use of available local material should be carefully considered. How this can be accomplished is excellently illustrated by the procedure for obtaining properly designed mixtures as described by G. A. Rahn of the Pennsylvania Department of Highways (179).

The soil of the original road was taken to the laboratory and on test was found to be a silt loam. Then it was found that stone screenings were available near the location of the road for use as

admixture. Next, the amount of admixture was applied to the old road material and worked in until tests disclosed that the proper amount had been added. Sometime later, calcium chloride was applied to the surface. The photograph (Fig. 182) shows the manner in which the stabilized road came through the winter in comparison with the side roads of the same material as the main road before stabilization. Sample 23 (Table 41) is representative of the mixture used by Rahn.

Typical Examples of Subgrade Soils.—Table 42 shows the grading and the constants of a number of samples representative of members of the eight uniform subgrade groups.

Sample 1, representative of a stable soil in South Carolina, has both the grading and the constants of the group A1 soils.

Sample 2, representative of soil serving satisfactorily as fill in Escambia Bay, Fla., has the constants but not the grading of A1 material.

Its low centrifuge moisture equivalent of 12 indicates that the slight excess of clay above that generally indicating A1 mortars is not likely to prove detrimental; and since the coarse sand in which this soil is deficient functions primarily to furnish the hardness required in wearing surfaces, this soil would probably prove highly stable as a subgrade.

The same is true for both sample 3, which was taken from a soil serving as a road surface in South Carolina, and sample 4, which was taken from a soil moderately stable when used as an untreated road surface in Madison County, Virginia, and highly stable when covered with bituminous surface treatments.

Sample 5 represents the nonplastic variety of the Florida shell rocks. Tests performed on small beams of this material disclosed that when thoroughly dry this variety of lime rock has practically no cohesion. Therefore, the stability of this material when used as a road surface must be due to the cohesion furnished by capillary pressure.

Sample 6 was obtained from a Florida sand which serves excellently as subgrade, when prevented from flowing laterally, for relatively thin road surface. Sample 7 was obtained from a California sand which serves excellently as a subgrade for concrete pavements $4\frac{1}{2}$ in. thick. Sample 8 was taken from a Minnesota sand which becomes highly stable when treated with bituminous materials possessing penetrative properties in high

TABLE 42.—MECHANICAL ANALYSES AND CONSTANTS OF TYPICAL SUBGRADE SOILS

Sample number	Sub-grade group number	Source, state	Physical characteristics, material passing No. 40 sieve					Mechanical analysis				
			Liquid limit	Plasticity index	Shrinkage limit	Shrinkage ratio	Moisture equivalent	Particles larger than 2.0 mm.	Particles smaller than 2.0 mm.			
									2.0 to .42 mm.	.042 to .005 mm.	Smaller than 0.005 mm.	Smaller than 0.001 mm.
							Centrifuge	Field				
1	41	South Carolina	17	4	15	1.9	13	14	57	39	16	8
2	42	Florida	19	6	17	1.8	12	15	19	51	18	12
3	42	South Carolina	20	7	18	1.8	13	16	7	72	6	16
4	42	Virginia	25	7	21	1.7	14	22	3	53	16	13
5	42	Florida	25	0	19	27	0	39	16	10
6	43	Florida	11	0	5	16	0	57	3	5
7	43	California	19	0	3	17	0	15	1	2
8	43	Minnesota	20	0	3	21	0	82	1	1
9	44	Florida	38	12	24	1.6	37	30	3	95	1	0
10	44	New Hampshire	27	0	27	1.6	23	27	0	26	42	2
11	44	Minnesota	38	13	22	1.6	35	30	1	21	52	4
12	45	Minnesota	36	9	28	1.5	40	31	0	14	27	40
13	45	New Hampshire	37	11	33	1.5	41	35	0	6	60	2
14	45	Oregon	43	9	37	1.3	38	41	3	29	60	0
15	45	Maryland	81	19	60	0.9	83	87	0	29	42	12
16	46	Missouri	55	36	13	1.9	55	36	0	11	57	16
17	46	District of Columbia	57	40	12	1.9	45	28	0	12	30	27
18	46	Virginia	132	101	11	1.9	178	33	16	25	6	88
19	47	Missouri	51	29	17	1.7	36	39	0	5	38	78
20	47	Colorado	42	24	14	1.9	35	34	0	7	55	24
21	47	Minnesota	71	42	14	1.8	69	43	0	0	45	21
22	47	Mississippi	112	82	9	2.0	116	52	2	9	43	22
23	48	Virginia	265	0	141	0.5	263	265	0	4	15	18
24	48	Minnesota	445	0	157	0.3	395	440	0	0	0	0

degree and covered with a thin application of granular material.

In spite of its greater fineness, the quartz sample (Table 27) has constants similar to those of samples 6, 7, and 8.

Sample 9 was composed of marl from Florida. Marl when kept dry and not subjected to frost action serves excellently as base-course material. Sample 10 was obtained from a New Hampshire silt which has heaved detrimentally under frost action. Sample 11 represents soil in the Minnesota frost-boil area which heaves under frost and loses stability during the spring thaws.

The similarity of the talc (Table 27) to samples 9 and 11 should also be noted. The loess (Fig. 72, Chap. VIII) is an A4 soil.

Sample 12 was composed of silt found in Saint Peter, Minn., which, according to F. C. Lang, heaved several feet during the winter of 1928-1929 (see Fig. 60, Chap. VII). Sample 13 was obtained from New Hampshire silts which have been found to heave in appreciable amount under frost action. Sample 14 was composed of Oregon silt which has been observed to undergo important frost heave. Sample 13 contains mica, while sample 14 contains an appreciable amount of organic matter.

Sample 15 was obtained from Maryland soil containing diatoms in appreciable amount. Soils of this character, especially when their shrinkage limits are equal to or greater than 50, are almost sure to produce pavement failure because of their high porosity. Sample 15, it will be noted, had the high liquid limit, shrinkage limit, and moisture equivalent of the diatom sample (Table 26).

Sample 16 was obtained from a colloidal-clay soil which has proved troublesome because of sliding in a fill in Missouri. Sample 17 was composed of colloidal clay furnished by a survey of the soil existing under the Potomac River at Washington, D. C. This soil was considered unfit for use as hydraulic-fill material. Sample 18 represents a very highly colloidal-clay soil productive of landslides in Virginia. This soil contains about 30 per cent of material so fine that it remains in suspension for weeks. If located on an impervious undersoil, this soil, when in a soft condition, acts as a lubricant facilitating the sliding of the upper soil layers.

Sample 19 was taken from an expansive clay on which a concrete pavement in Missouri cracked in appreciable amount during the setting period of the concrete. Sample 20 was

obtained from an expansive clay in Colorado on which similar cracking occurred in the pavement.

Sample 21 was composed of gumbo from the Red River valley, Minnesota, which does not suffer detrimental frost heave and which is stabilized when oil treated and covered with granular material.

Sample 22 represents a flocculated, highly colloidal soil from Mississippi. Only the absence of waterlogging with a centrifuge moisture equivalent as large as 116 prevents this soil from being grouped with the A6 subgrades. This soil, which contains both lime and gypsum, supports concrete slabs which have warped in detrimental amounts. Soil of this character cannot be used efficiently in fills, and in cuts it should be separated from concrete pavements by a good topsoil-base course at least 2 ft. thick.

Sample 23 was composed of a peat that failed to support a dry-land fill in Virginia; and sample 24 from a peat that has proved inadequate to support fills in Minnesota.

Samples 23 and 24 illustrate the very high water-absorptive properties characteristic of the peat soils that have not yet reached the colloidal state by decomposition. That these two soils have not yet reached this state is indicated by the fact that their plasticity indexes are equal to zero and by the absence of waterlogging in the centrifuge test.

Problems

1. What routine test result indicates the moisture content of natural-soil and old-soil sediments?

2. Discuss how the natural density of clay undersoil may influence design of the foundation.

3. How does the height of a fill influence the selection of materials for use in the construction of the fill?

4. Describe a simple test for use in determining the suitability of soil when fills are to be constructed by the hydraulic method. Under what conditions should only permeable material be used in the hydraulic method of construction?

5. Describe simple methods which may be used for determining the moisture content and degree of densification of soils being placed in embankments.

6. What are lime rocks? Where do they principally occur? What tests best disclose their suitability?

7. What are caliches? Where do they principally occur? What tests are used principally to disclose their suitability?

8. Describe the procedure used to provide road soils with the desired properties.

CHAPTER XVII

MISCELLANEOUS PROPERTIES AND TESTS

The great variety of problems met in soil work requires the use of tests and apparatus not always available in the routine laboratory. The more common of these methods of examination are briefly described to round out the picture of the comprehensive scope of the field of soil engineering.

Elasticity Differs from Expansion.—Rebound of soils due to elasticity is distinguished in several ways from expansion due to increase in moisture content. Elastic rebound upon removal of load characteristic of the A5 soils is due to the presence of particles that have spongy or elastic properties. Expansion on removal or reduction in load due to increase in moisture content characteristic of the A6 and A7 groups is caused by the capillarity of the soil. Such expansion occurs only in soils having access to water and principally in a vertical direction. Elastic rebounds, in contrast, occur in both wet and dry soils and take place both laterally and vertically.

The effect of elasticity in producing vertical rebounds is determined by loading and unloading both wet and dry soils in confined state in a manner similar to that used in the compression test (see Fig. 84) but with the soil having no access to water. The effect of elasticity in producing both lateral and vertical distortions is disclosed by tests performed on unconfined samples.

The elastic properties of the muck soil in confined state are indicated by the curves in Fig. 183. According to this figure, the air-dried muck, when loaded and unloaded without access to water, was compressed to a pore ratio of 1.70 under a load of 3.2 tons per square foot and on complete removal of load expanded to a pore ratio of 1.77. According to these determinations, the elasticity of the muck soil is sufficient to cause a 20-ft. layer of the dry material under a load of 3.2 tons per square foot to assume

a thickness of $\frac{1 + 1.77}{1 + 1.70} \times 20 = 20.52$ ft. on removal of load.

Under similar conditions a layer of wet muck soil 20 ft. thick, under a load of 3.2 tons per square foot, would, according to Fig. 183, become 20.68 ft. thick on removal of load, owing to the elasticity of the material.

The elastic properties of unconfined soils are disclosed by consistency tests performed on cylindrical soil samples. These tests also yield information on the ultimate bearing value (yield point) and the rate of settlement of unconfined soils. The

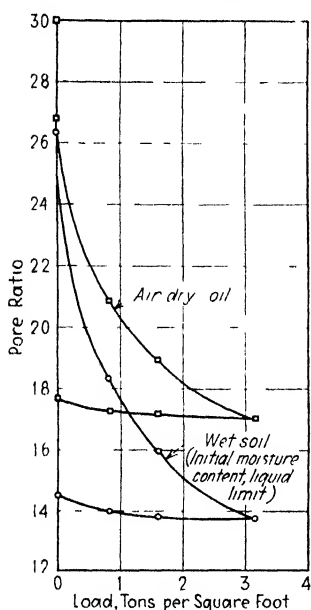


FIG. 183.—Special compression-test data for muck soil.

apparatus and type of results furnished by these tests are illustrated in Fig. 184.

In these tests the load is changed (increased or decreased) by equal increments at equal time intervals, and observations are made of the rate and amount of compression or expansion corresponding to each application or removal of load.

If in such a test a new increment is added to the load, an almost instantaneous compression follows. As time goes on, the rate of compression decreases until finally equilibrium is reached. On the other hand, on removing the same load increment the specimen almost instantly expands, the rate of expansion decreasing until finally movement ceases.

These phenomena are characteristic of the effect of pressure below the yield point, or ultimate bearing value, and correspond to the effect of pressures on solid bodies.

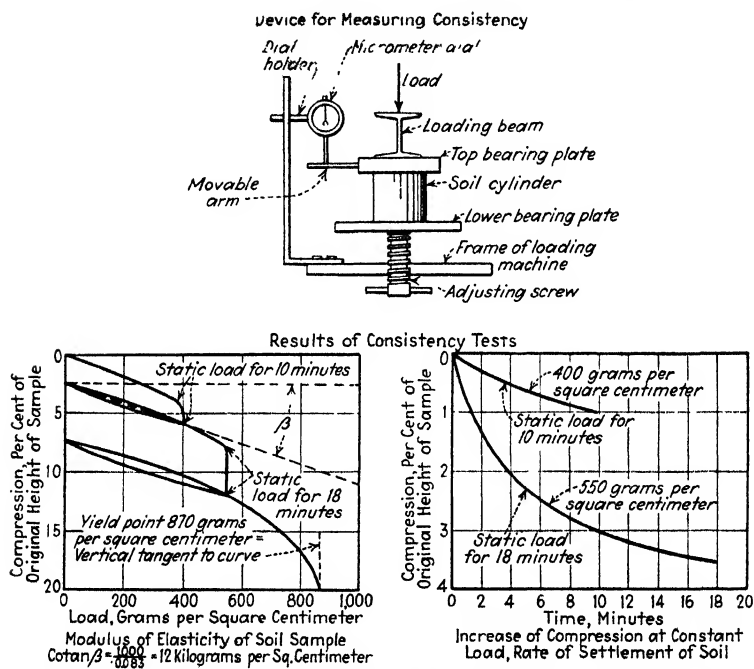
Two facts brought out by these results are: (a) The application of a pressure on clay always involves a viscous flow, regardless of whether the pressure be below or above the yield point; (b) if the pressure is below the yield point, the speed of flow decreases at constant pressure until it finally equals zero.

The maximum pressure below which a state of final equilibrium can still be reached corresponds to the yield point. Hence the time-compression curves for loads below the yield point approach

a horizontal tangent, while the corresponding curves for loads above the yield point have no such tangent.

In Fig. 184 (lower left) the yield point corresponds to the abscissa of the vertical tangent to the load-compression curve, because the load-compression curve joining this tangent means infinite increase in compression at no increase in load.

Tests cannot be carried to the point where the compression curve becomes vertical, but the position of this point can be



estimated with a sufficient degree of accuracy. In Fig. 184 (lower right) the increases of compression at different constant loads are shown.

The degree of elasticity of the samples is indicated by the reversible part of the compression and can be expressed by the modulus of elasticity, as shown in Fig. 184 (lower left).

In foundation work, elasticity is a soil property not often requiring much consideration. Loads are usually applied very soon

after excavations are completed, and, furthermore, the dead loads usually exceed the live loads appreciably, and there is little chance for rebound. In highway work where loads are increased and decreased at frequent intervals, or where loads and impacts far exceeding those that the structure will normally carry are used to produce a high degree of consolidation, elasticity becomes an item of major importance.

Either a macadam or a concrete surface may be seriously damaged by attempts to consolidate elastic subgrades before pavement construction. After the thorough rolling which benefits the compressible subgrades, a subgrade of the elastic type, if it possesses cohesion, is likely to retain a certain degree of compaction. A slight wetting under these conditions may cause non-uniform rebound of the subgrade. Movements of heavy material trucks and mixing apparatus adjacent to pavements laid on elastic cohesionless subgrades may cause distortions of the soil supporting the freshly laid concrete sufficient to produce pavement cracking.

Finally, elasticity may cause embankments, which were constructed to a satisfactory degree of consolidation, to expand and decrease in density to a point where they no longer are safe for the purpose for which they were designed.

Critical Moisture Content.—The critical moisture content previously defined and discussed in Chap. V (page 102) may be determined either by load-deformation tests performed on unconfined samples by means of the apparatus shown in Fig. 184 or by load-penetration tests performed on confined samples as illustrated in Fig. 185. The type of results furnished by the load-penetration tests is illustrated in Figs. 186 and 187. Figure 186 shows the relation of load to penetration for the same soil at different moisture contents ranging from 16.4 to 30.6 per cent. Figure 187 shows the relationship between the moisture contents and the corresponding penetrations under a constant load of 0.5 lb. per square inch. It will be noted that for increases in moisture content up to about 24 per cent the penetration increases very slowly, while for similar increases in moisture content above 24 per cent the penetration increases at a much greater rate. The intersection of the moisture content-penetration curve for the moisture contents up to 24 per cent with that for the moisture contents above 24 per cent indicates the moisture content at

which the abrupt change from the semisolid to the plastic state occurs, which by definition is the "critical moisture content."

While not a routine test, the determination of the critical moisture is of considerable importance, because it assists in an

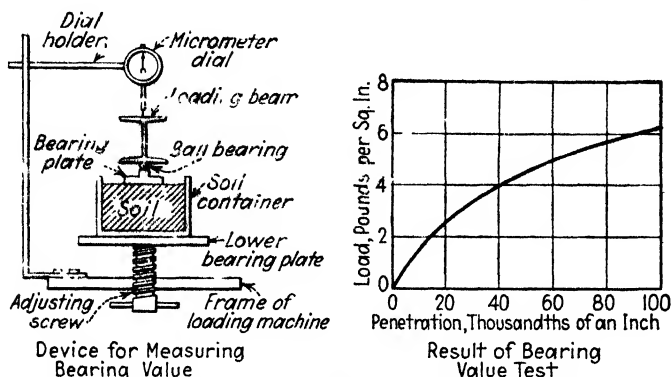


FIG. 185.—Apparatus and type of results furnished by bearing-value tests.

understanding of the significance of the plasticity tests and indicates the extent to which the moisture content of soils must be reduced in order that they may be stabilized by drainage.

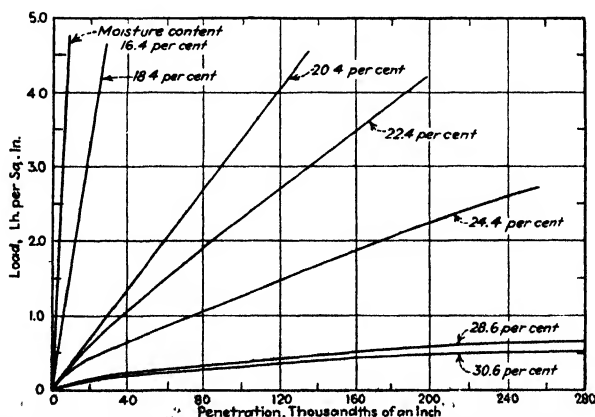


FIG. 186.—Relation of load to penetration for a soil at different moisture contents.

Crushing Strength and Angle of Internal Friction.—Bearing-value tests performed on dried cylindrical specimens of soil by means of apparatus such as is shown in Fig. 184 may be used to

determine (a) the crushing strength and (b) the angle of internal friction ϕ . The latter is given by the formula

$$\phi = 2\alpha - 90^\circ \quad (78)$$

in which α = the angle of fracture.

Tensile Strength.—Tensile-strength tests are usually made on briquettes similar to those used in testing Portland cement and molded in a standard manner (Searle). They may be made in either a dry or a wet condition. Hind (180) has found that “for accurate work it is necessary to maintain the specimen in a saturated atmosphere of constant temperature and humidity during the tests.” Great variations frequently occur in determining the tensile strength so that a considerable number of tests

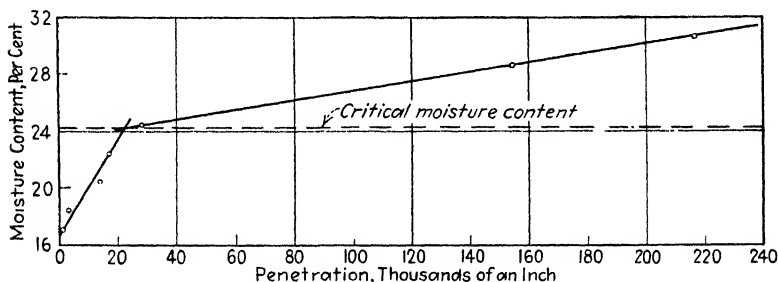


FIG. 187.—Relation of moisture content to penetration under a load of 0.5 pound per square inch.

should be made to obtain an approximately accurate average result.

John H. Griffith (17) reports the determination of the tensile strength of soil columns in the undisturbed state in the field. In these tests the earth was carefully trimmed away from the specimen, leaving it standing in its natural condition. A wooden form was clamped about the specimen, and a vertical pull exerted by means of a chain hoist. The force required to rupture the specimens was measured, and cross section of break determined. Large variations in the determined tensile strengths were observed.

Rate of Slaking.—The slaking test consists essentially in a determination of the rate at which a soil specimen disintegrates when immersed in water. The rate of disintegration depends upon such factors as the affinity of the soil for water, the amount of

expansion of the soil when wetted, the speed with which the water penetrates the soil, and the character of the cracking produced by unequal expansion of the sample. The test gives an indication of the relative resistance of different soils to erosion and may also be used to determine the effect of different admixtures for waterproofing a given soil. Also, tests performed on undried specimens are a measure of the extent to which the capillarity of the soil is satisfied by the moisture present in the sample. The test is more or less qualitative in nature, and neither a standard procedure nor definite limiting values have been established.

In the original procedure used by the Bureau of Public Roads a dried cylindrical specimen of soil about 1 in. in diameter and 1 in. in height was placed on a 1-in. diameter brass ring attached to a supporting device and completely immersed in water. The time in minutes required for the specimen to disintegrate sufficiently to fall through the ring was taken as the slaking value of the soil. In more recent tests a coarse wire screen has been substituted for the brass ring.

When the foregoing procedure is used it is sometimes difficult to determine the end point because of the turbidity of the water produced by suspended soil material. This difficulty may be overcome by a proposed method in which the soil specimen is suspended and but partially immersed in water in a container placed on one platform of a balance, and the rate of disintegration determined by observing the time required for a given percentage of the specimen to be deposited in the container.

In another variation of the slaking test used by different investigators the soil is molded into a briquette similar to those used in determining the tensile strength of Portland-cement mortar. The entire briquette, with the exception of a narrow strip about the center, is waterproofed by coating with paraffin. The briquette is suspended in water in a vertical position to a depth such that the uncoated portion is completely immersed. The time required for the lower half of the briquette to separate from the upper is taken as the slaking value of the soil.

Measurement of Earth Pressures.—A soil-pressure cell was developed by A. T. Goldbeck in collaboration with E. B. Smith principally to measure pressures within earth fills (181, 182, 183). The construction of the cell is plainly indicated in the cross section shown in Fig. 188. A $\frac{1}{8}$ -in. pipe containing insulated lead wires

is connected to the control and recording apparatus (Fig. 189), which consists of a compressed-air tank, control valve, gages, and electrical connections including a dry cell and electric light. Two accurate pressure gages are used, one graduated to read from 0 to 30 lb. in 0.2-lb. increments, and the other up to 100 lb. in 1 lb. increments.

When in operation the cell is placed so that the movable side *B* is in contact with the earth fill in the direction necessary to give

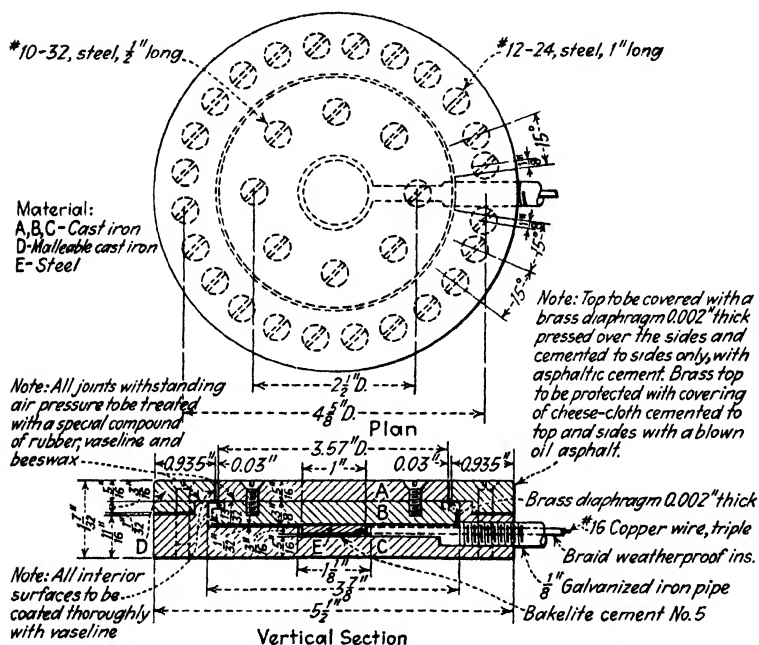


FIG. 188.—Diaphragm cell for determining soil pressure.

the desired component of pressure. The control valve on the pressure tank is opened, air is admitted very slowly into the pressure cell, and at the instant when electrical contact is just broken between *B* and *E*, as indicated by the extinguishing of the small electric light, the reading of the gage pressure is read. The air pressure as read from the gage equals the earth pressure acting on the cell.

Hveem Stabilometer.—A stabilometer developed by F. N. Hveem (184) consists of an apparatus in which the defor-

mation of a specimen of a bituminous mixture under load is transmitted by means of a liquid to a pressure gage. The apparatus consists essentially of a cylinder having part of its

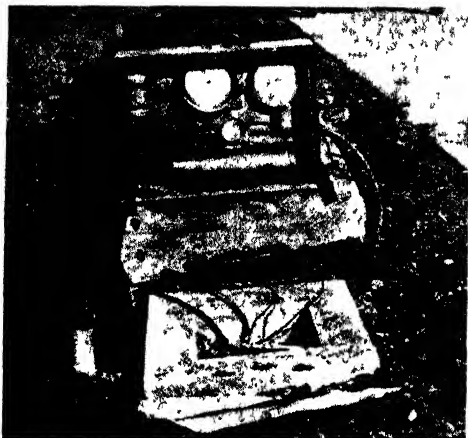


FIG. 189.—Control and recording apparatus.

inner wall cut so that when a rubber diaphragm is securely fastened to each end, a chamber will be formed. This chamber is then filled with a suitable liquid after the specimen has been

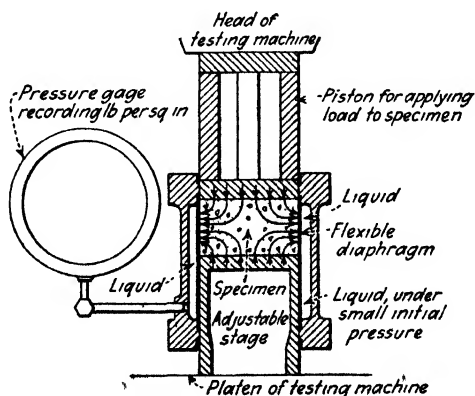


FIG. 190.—Hveem stabilometer.

inserted, and an initial pressure is applied such that the rubber diaphragm will make close contact with the sides of the specimen (Fig. 190). Pressure is then applied to the specimen at a uniform

rate, and the resulting deformation is indicated by the pressure-gage readings.

Water-asphalt Preferential Test (184).—This test is intended to indicate the affinity of fine material, passing a 200-mesh sieve, for water or asphaltic liquids. The apparatus consists of an electric stirrer with speed control, a control-temperature water bath, and an 8-oz. wide-mouth bottle with screw cap.

The test procedure is as follows:

A 50-ml. sample of heavy fuel oil (60 to 70 per cent asphalt) heated to 140°F. is placed in the sample bottle, and 10 grams of filler material passing the 200-mesh sieve is added. The bottle is then placed in the hot-water bath, maintained at 140°F., and the mixture stirred for 5 min. with the mechanical mixer revolving at a speed of 1,500 r.p.m. After 5 min., 100 cc. of distilled water at 140°F. is added, and the mixture again stirred for 5 min. The mixture is then allowed to settle until the water becomes clear. The clean material in the bottom of the bottle is estimated and reported as the percentage of the total separated. A satisfactory material is indicated when the fine particles retain the oil and do not separate as clean particles in the bottom of the bottle. According to Stanton and Hveem, a tolerance up to 25 per cent is allowed.

This test is considered by them as being but a supplement to the swell test.

Swell Test (184).—This test was devised to show the effect of water in producing swell in soil mixtures. It consists of making a briquette of standard dimensions by compressing the bituminous mixture under a standard load. The briquette, without being removed from the mold, is then placed on a stand, and a perforated plate placed on top. A micrometer dial is then adjusted in contact with a stem attached to the perforated disk, and the reading noted. Five hundred cubic centimeters of water is then poured into the upper part of the mold, and after 24 hr. another reading is taken. The swelling of the specimen is recorded as the difference between the two readings.

Stake Method for Determining Stability of Subgrade. D. P. Krynine (20) is the originator of a method for evaluating the stability of the subgrade. In this test a rigid metal stake $1\frac{1}{4}$ by $1\frac{1}{4}$ in. in cross section and 41 in. long is driven into the soil to a depth of 12 in. The top of the stake is attached to the axle

of a car by a wire 12 ft. long. Upon being pulled by the car the stake turns about a point about two-thirds of the distance of the depth to which the stake has been driven. Cracks limited by a circle are formed on the surface of the earth; finally a conic body is pushed out. A scale introduced in the apparatus records the tension in the wire, and the maximum reading is recorded.

Three forces resist the pushing out of the cone:

1. The cohesion c acting along the surface of the cone.
2. The friction f forming the angle of friction ϕ , with normals to the surface of the cone.
3. The weight of the cone W

The vertical resultant R of the cohesive and frictional forces and the weight W are the forces to be overcome in order to push out the cone.

Let S = reading of scale.

a = arm of force S with respect to center of rotation of stake at pushing out of cone.

r = radius of base of cone.

Then the condition of equilibrium immediately preceding the destruction of friction would be

$$Sa = (R + W)r \quad (79)$$

Neglecting the weight of the soil,

$$R = a \frac{S}{r} \quad (80)$$

This means that the shearing resistance is proportional to the ratio S/r or to the ratio $S/2r$. Therefore the ratio of the reading on the scale to the average diameter of the hole produced on the ground may be called the shearing characteristic of the subgrade.

Chemical Analysis of Soils.—While the agricultural chemist interprets the chemical analysis in the light of soil fertility and productiveness, a knowledge of the chemical composition furnishes the soil surveyor a basis for expressing the characteristics of soils in relation to their environment. The chemical analysis, either complete or partial, may also serve the engineer by indicating the amount and kind of different deleterious substances contained in the soil.

A routine chemical analysis is generally confined to the quantitative determination of the following components: silicon, iron,

aluminum, titanium, calcium, magnesium, potassium, sodium, manganese, phosphorus, and sulphur present as oxides; nitrogen, organic matter, combined water, and carbon dioxide present as carbonates. Instead of a complete routine chemical analysis, a determination of the total percentages of silica, aluminum, and ferric oxide or of the exchangeable bases and total exchange capacity may be sufficient for many purposes.

Depending upon the information desired, the chemical analysis is made on the entire soil, on certain fractions only, on the soil solution, or on the water or acid extract. For detailed methods of chemical analysis of soils the reader is referred to publications listed in the bibliography (45, 185, 186).

Hydrochloric Acid Reaction.—The amount of carbonates in soil may be determined qualitatively by adding a few drops of hydrochloric acid to a small sample. The violence of the reaction is proportional to the amount of carbonates present.

Dye Adsorption.—E. C. E. Lord of the Bureau of Public Roads (187) has devised a test which consists essentially of filtering a solution of a dye such as basic aniline dye (methyl violet) through a soil and estimating the amount adsorbed from the decoloration of the filtrate. The amount of dye solution (1 part dye to 1,000 parts distilled water) decolorized by 1 gram of soil is termed the "dye adsorption number."

The dye adsorption number is a rather complex function of the specific surface of the soil material, the physicochemical character of the surface, and the quantity of electrolytes present in the soil. Since the greatest part of the specific surface is made up of the surfaces of the colloidal fraction, the dye adsorption number can be said to depend principally on the amount and character of the soil colloids. At present it is not possible to evaluate the relative influence of these two factors.

Hydrogen-ion Concentration.—The degree of acidity or alkalinity of suspensions of soil in water may be measured by one of two methods: the electrometric or the colorimetric. The electrometric method consists in measuring the potential developed by an electrode in a soil suspension as compared with that developed by a standard electrode. The colorimetric method consists in treating dilute suspensions of a given soil with a series of indicator solutions, each of which gradually changes color when acted upon by solutions of varying degrees of hydrogen-ion con-

centration. The degree of acidity or alkalinity is expressed as a pH value, which indicates the amount of ionized hydrogen present.

A number of indicators covering the pH range likely to be encountered in soil work, together with the pH range and color change (acid to alkaline) of each, are given in Table 43 (45).

The colorimetric method involves two steps: (a) the selection, by trial, of the particular indicator having the pH range within which the pH of the suspension falls; (b) the determination of the pH of the suspension by comparison of the final color of the indicator used with color standards indicating definite pH values. For accurate colorimetric determinations, the suspension should be clarified by centrifuging or dialysis.

TABLE 43.—INDICATORS USED IN SOIL WORK

Indicator	pH range	Color change, acid to alkaline
Bromphenol blue	3.0 to 4.6	Yellow to blue
Bromcresol green	3.8 to 5.4	Yellow to blue
Bromphenol red	5.2 to 6.8	Yellow to red
Bromthymol blue	6.0 to 7.6	Yellow to blue
Cresol red	7.2 to 8.8	Yellow to red
Cresol phthalein	8.2 to 9.8	Colorless to red

Heat of Wetting.—The heat of wetting is the amount of heat that is developed in the wetting of the dry soil by water. It is believed by Freundlich (44) to be the heat of adsorption of vapors at their saturation pressure. It is considered as the heat developed when the first film of molecular dimensions is formed on the soil surface. When silica, sand, or glass is brought into contact with water at approximately constant temperature, the development of heat at 7°C. is approximately 0.00105 cal. per square centimeter of surface wetted.

The apparatus used in determining the heat of wetting of soils consists of a flask and a thermometer. Water is placed in the flask, and the thermometer inserted and allowed to remain until the temperature reading remains constant for 1 min. The dry soil is then added, and the thermometer read at minute intervals, the mixture being stirred before each reading with the thermometer, until the temperature becomes constant.

The heat of wetting is calculated from the formula

$$Hw = (T_2 - T_1)V \quad (81)$$

where Hw = heat of wetting, calories.

T_1 = temperature of water before soil is added, degrees centigrade.

T_2 = final temperature of soil-water mixture, degrees centigrade.

V = (mass of water plus soil) SH , specific heat of water.

Crystallization of Soil Chemicals.—Closely allied to the physical aspects of the solidification of moisture on freezing is the crystallization of soil chemicals from solution. While the growth due to such crystallization is too small to contribute appreciably to the heaving of soils, it may become large enough in the pores of rock and similar materials to produce destructive effects.

Especially in studies of the durability of concrete, tile, and similar materials has the phenomenon of crystal growth been of interest. Disintegration of such materials may be of three types: chemical, part chemical and part physical, and physical.

The purely chemical type results from such causes as the action of sea water or the effects of alkali soil, of sulphates of sodium and magnesium, of various acids found in ground waters, sewage, etc., or of the many chemical compounds of a greater or less stability which may, under certain conditions, be set up in the hydration of cements (188). The detrimental chemical, whether in soil or water, must be present in amount sufficient to produce a readily discerned action. Such action involves base exchange and would be expected to begin where the concrete is in contact with the detrimental agency and progress from this location toward the interior of the structure. In road slabs, detrimental action of this type would begin at the bottom and progress upward.

In the physicochemical type the active chemical is assumed either to enter the slab in solution owing to capillarity or to be dissolved by contained moisture from some location within the structure. It is then deposited at some location on the interior of the structure where the rate of percolation changes or at the surface where evaporation occurs.

Such action would serve to explain why piers, piles, and the like in water with chemical content obviously too low to injure

the completely immersed portions of the structure may, by continued capillary flow and evaporation, accumulate enough chemical just above the water line to be detrimental; also to explain the type of deterioration of pavement slabs which begins at the top and works downward.

The purely physical type results from the growth of crystals without chemical action. The crystals may be formed by the freezing of water during low temperatures or the crystallization of dissolved chemicals due to the evaporation of the liquid carrier (189). This, like the physicochemical type, can be expected to begin at the surface of freezing or evaporation and progress toward the interior of the slab.

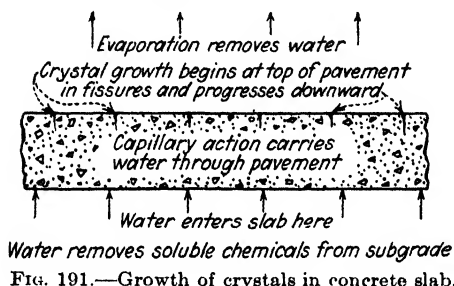


FIG. 191.—Growth of crystals in concrete slab.

Figure 191 illustrates the manner in which soil solutions are assumed to travel through road slabs in order that crystals may be deposited at the tops of the slabs.

The rate of travel depends upon the permeability, and this is controlled by the character more than the percentage of voids in the mortar. Especially does the presence of fine cracks, almost invisible to the naked eye but affording, nevertheless, more or less continuous channels, increase the permeability.

That disintegration progresses more rapidly in natural rocks which contain cracks and laminations than in those which do not is well known. In experiments on the phenomenon of crystal growth, Taber (190) found that such materials as brick, cement mortar, and even pottery also were less resistant when containing small fissures.

Crystals of Water-soluble Chemicals.—Figures 192, 193, and 194 show photomicrographs of crystals of well-known chemicals. Crystals of some chemicals differ in character owing to difference in the strength of the solutions from which they were obtained.

This is true of crystals from sodium hydroxide (Figs. 192A and 192B); from sodium sulphate (Figs. 192E, 192F, and 192G); and from calcium sulphate (Figs. 193D and 193E). This was not found true for crystals from magnesium sulphate solutions.

The calcium sulphate, or gypsum, crystal (Fig. 193C), has a maximum linear dimension of about 2 in. It was obtained with a number of similar crystals by W. I. Watkins from the

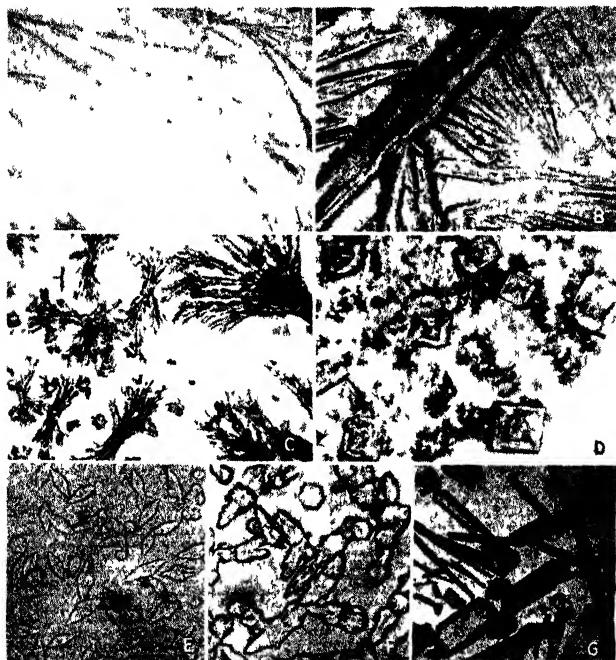


FIG 192 — Typical sodium crystals.

Red River valley, Minn. Samples of the gumbo soil from this location were examined in connection with studies of the warp of concrete pavements. The alkali (Fig 194C) was made up synthetically to represent the water at Billings, Mont., the action of which on concrete has been extensively studied (191). Here, and also in Fig. 194E the crystals have the typical sawbuck form of crystals from dilute sulphate solutions.

The crystals (Figs. 194F and 194G) are from two samples of the same cement mortar. The sample of Fig. 194G, however, was

subjected to the action of a synthetic alkali solution in contrast to the sample 194F which was not subjected to such action.

The mortar furnishing the crystals in Fig. 194G was obtained from that portion of a test sample located about 2 in. above the portion of the sample in contact with the chemical solution. The treatment had progressed for a period of about 2 months.

The hexagonal crystals from the treated mortar (Fig. 194G) were probably furnished by the same sulphates as the sawbuck



FIG. 193 —Typical calcium crystals.

crystal. To date, the hexagonal form of the sulphate crystal has been found only in cement mortars and concretes and might be due to the presence of sulphur in the mortar as well as in the penetrating chemical solution.

Crystal growth is quite erratic, occurring at times on one side of the pavement and not on the other and even in one portion of a slab and not in the remainder. Influencing variables include the amount of water used in the mixture, the manner of curing, and

the loss of moisture from the slab by evaporation due to atmospheric agencies such as temperature and wind and due to absorption by the subgrade soil.

Typical Crystals.—Crystals furnished by studies of the durability of concrete (192) in concrete pavements, culverts, retaining

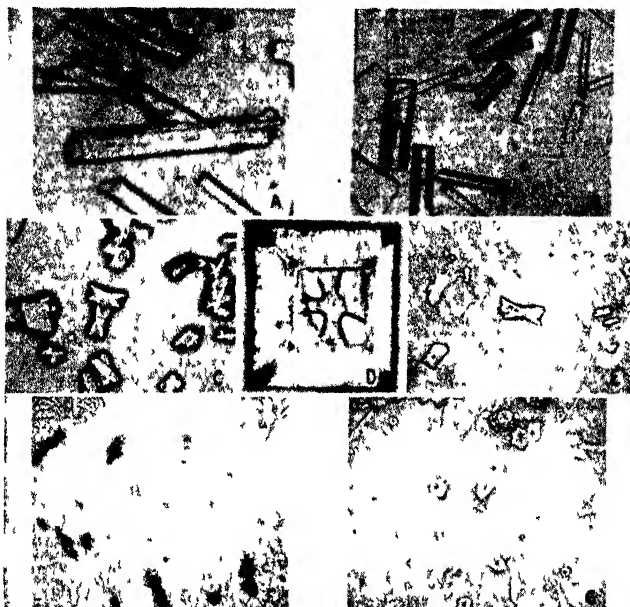


FIG. 194.—Miscellaneous crystals.

walls, piers, abutments, and side walls, shown in Figs. 195 and 196, are identified as follows:

Figure 195:

A. Soil under progressively scaled section of a road in Virginia. The fissure (Fig. 197) was in scale of section of this road.

B. Efflorescence on badly damaged tile wall, Buffalo, N. Y.

C. Disintegrated tile in damaged wall, Buffalo, N. Y.

D. Efflorescence on badly disintegrated back wall, Pennsylvania.

E. Water flowing over same wall D.

F. Example of amorphous calcium carbonate from soil in Pennsylvania.

G. From soil under fissured pavement, see H, in Georgia.

I. Salt crystals from concrete in badly cracked and checked road in Alabama.

J. From soil under disintegrated pavement, Missouri.

Figure 196:

- A. From deteriorated pavement slab, Alabama.
- B. From Red River Valley gumbo in Minnesota (see Fig. 193).
- C. From soil in Texas associated with pavement warping.
- D. From soils in California associated with pavement warping.

Corrosive Effect of Soil Chemicals.—The purely chemical type of deterioration is of interest particularly in studies of soil corrosion of metal pipes, culverts, bins, etc.

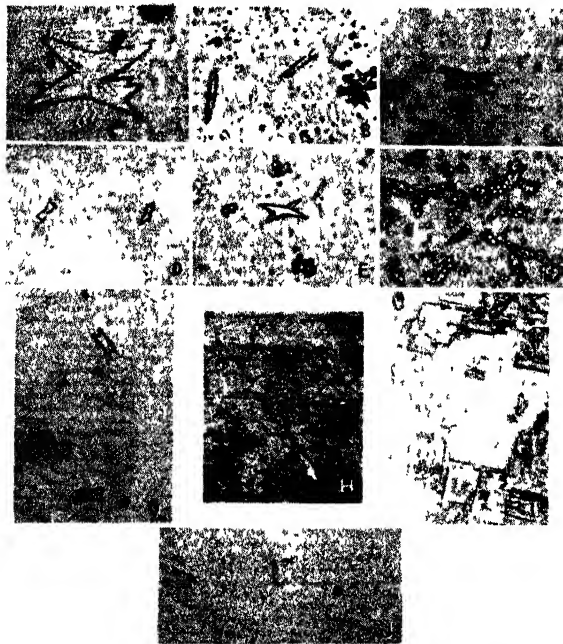


FIG. 195.—A-G, I, J, typical crystals from soils or structures; H, pavement source of crystals I.

The underground pipes in the United States have been estimated to have a length of 450,000 miles and a value of nearly \$6,000,000,000. The annual loss due to underground corrosion of pipe lines in the oil industry alone has been estimated at \$25,000,000.

In soil-corrosivity studies (80) measurements of several physical properties of soils which determine their permeability have been correlated with the pitting factor. The latter is defined as the ratio of the maximum depth of the pits to the average depth of penetration. These studies indicate that the more permeable the

soil, exclusive of sands, the greater the tendency for corrosion to occur. Thus greater corrosive action may be expected in silt soils than in clay soils or sands, other conditions being the same.



FIG. 196 — Typical crystals from soils and mortars

Clay soils are likely to be so impermeable that the rate of pitting in them will be very low.

Soil corrosion of metals is characterized by an uneven distribution of the corrosion and by the fact that most ferrous materials corrode at nearly the same rate. Corrosion of ferrous materials in acid soils is influenced largely by the degree of acidity of the soil. The corrosive effect of soil acidity is attributed to its action in preventing the formation of protective layers of ferrous and ferric hydroxides.



FIG. 197.—Deposit of chemical at edges of fissure in mortar. Length of fissure about $\frac{1}{16}$ inch (50 diameters).

The major cause of corrosion seems to be nonuniformity in the distribution of oxygen and moisture along the surface of the metal. The tendency of iron to go into solution is a function of the hydrogen-ion concentration (pH) of the soil solution if there is a rapid displacement of the solution in contact with the metal and no tendency to form a surface coating. In a solution with a pH of 8.0 no iron will be dis-

placed in the absence of oxygen. But with the presence of oxygen in the solution, iron will be displaced, and the pH will drop to 6.5.

The effect of the pH of the soil on its corrosive properties was noted in Chap. IV.

It has been found that the more corrosive soils have lower electrical resistance. This lower resistance is accounted for by the presence of higher percentages of water or of soluble salts. Thus determinations of the moisture content and identification of soluble salts by the microchemical method of analysis may be used to disclose the relative electrical resistance and, in turn, the corrosive properties of the soil.



FIG 198 —Apparatus used in the preparation of samples for microscopic examination

Microchemical Analysis (192).—A rapid qualitative test for the chemical constituents of soil solutions, ground water, or solutions of the soluble material in cement-mortar samples consists in drying a small portion of the solution on a glass slide and examining the residue with a microscope. Crystals so formed may be identified by comparison with photographs or drawings of crystals of known chemical composition. Figure 198 shows the apparatus used in the preparation of the solutions and slides. Photomicrographs of crystals of chemicals often present in the foregoing solutions are shown in Figs. 192 to 196 inclusive.

Problems

1. Enumerate four ways in which the rebound of soils due to elasticity differs from expansion due to increase in moisture content.

2. How is the effect of elasticity in producing vertical rebound determined?

3. A sample of air-dried soil when loaded in confined state without access to water was compressed to a pore ratio of 1.45 under a load of 3.2 tons per square foot and on complete removal of load expanded to a pore ratio of 1.62. If a layer of this soil in air-dry state were 15 ft. thick under a load of 3.2 tons per square foot, what would its thickness become on complete removal of load?

4. (a) What are the two principal facts brought out by the results of the consistency tests?

(b) How do the time-compression curves (as determined on unconfined samples) for loads below the yield point differ from those of the corresponding curves for loads above the yield point?

5. Discuss the possible effect of elasticity on the performance of (a) foundation soils, (b) subgrade soils, (c) embankments.

6. (a) Define "critical moisture content."

(b) What is its significance with respect to

1. The performance of clay binders?

2. The drainage of soils?

7. If the angle of fracture of a soil as determined by the crushing strength test is 60 deg., what is its angle of internal friction?

8. (a) What are some of the factors that influence the rate of slaking of dry-soil specimens?

(b) Describe one method for determining the slaking value of soils.

9. (a) Name six components of soil which are determined in a routine chemical analysis.

(b) Describe briefly the procedure used in the microchemical analysis of soils.

(c) How may the presence of carbonates in soils be determined qualitatively?

10. What are the chief factors upon which the adsorption of dye by a soil depends?

11. Describe briefly the colorimetric method for determining the hydrogen-ion concentration of soils.

12. If in a determination of the heat of wetting of a soil the temperature of the water used is 20°C., the final temperature of the soil-water mixture 21.4°C., and the weights of the water and the soil sample 100 and 5 grams, respectively, what is the heat of wetting of the soil, the specific heat of water being assumed as unity?

13. State the three types of disintegration that may occur in concrete slabs. Explain briefly how each type may occur.

14. Describe the growth of crystals in a concrete road slab when the subgrade soil contains soluble chemicals.

15. Upon what does the rate of disintegration occurring in the concrete slab (problem 14) depend?

APPENDIX I

CONVERSION TABLES

(From Smithsonian Tables)

METRIC TO U. S.

Metric	U. S.	
	Inches:	Feet:
1 millimicron (0.001 micron).....	0 000000039	0.0000000033
1 micron (0.001 millimeter)	0 000039	0.00000328
1 millimeter (0.001 meter).....	0 03937	0.00328
1 centimeter (0.01 meter).....	0.39370	0.0328
1 meter.....	39.37	3.28
	Square inches:	Square feet:
1 square millimeter.....	0 0015	0.0000108
1 square centimeter.....	0 1550	0.001076
1 square meter.....	1550	10.764
	Cubic inches:	Cubic feet:
1 cubic centimeter.....	0.0610	0.000035
1 liter (1,000 cc.).....	61.02	0.03531
1 cubic meter.....	61,023	35.314
	Pounds:	Ton:
1 gram.....	0.002205	0.0000011
1 kilogram.....	2 205	0.0011
	Pounds per square foot:	Tons per square foot:
1 gram per square centimeter.....	2.048	0.00102
1 kg per square centimeter.....	2048	1.024
1 kg per square meter.....	0.2048	0.000102

U. S. TO METRIC

U. S.	Metric		
	Millimeters:	Centimeters:	Meter:
1 inch.....	25.4	2.54	0.0254
1 foot.....	304.8	30.48	0.3048
	Square mil- limeter:	Square cen- timeters:	Square meter:
1 square inch.....	645.1	6.4516	0.00065
1 square foot.....	92903	929.03	0.0929
	Cubic cen- timeters:	Liters:	Cubic meter:
1 cubic inch.....	16.387	0.0164	0.000016
1 cubic foot.....	28300	28.3	0.0283
1 gallon.....	3785	3.785	0.0038
		Grams:	Kilograms:
1 pound.....	...	453.6	0.4536
1 ton.....	...	907,200	907
	Grams per square cen- timeter:	Kilograms per square cen- timeter:	Kilogram per square meter:
1 pound per square inch.....	70.307	0.0703	0.00000703
1 pound per square foot.....	0.48824	0.00049	0.00000005
1 ton per square foot.....	976.48	0.9765	0.000098

APPENDIX II

NOMENCLATURE

Chapter II

- E* Porosity in percentage of soil mass.
- e* Pore or voids ratio to solids.
- w* Moisture content in percentage by weight of solids.
- G* Absolute specific gravity.
- G₀* Bulk specific gravity.
- V* Volume of soil mass.
- V_s* Volume of solids.
- V_e* Volume of pores or voids.
- D_b* "Bulk" density.
- D_s* "Absolute" density.
- s* Diameter of particles.
- n* Coefficient of viscosity in poises.
- L* Depth of settlement.
- g* Gravitational constant.
- v* Velocity of particle settlement.
- G₁* Specific gravity of suspending medium.
- FL* Flocculation limit.
- FR* Flocculation ratio.
- H* Hydrometer reading.
- W_s* Weight of solids dispersed.
- x* Grams of soil in suspension.
- W_w* Cubic centimeters of distilled water in the suspension.
- ΔH Correction in hydrometer reading for change of temperature from 67°F.
- f* Correction coefficient for change of specific gravity from 2.65.
- P* Percentage of originally dispersed soil remaining in suspension.
- es* Effective size.
- uc* Uniformity coefficient.

Chapter III

The nomenclature used by Shepard in his original discussion of the resistivity and seismic methods of subsurface exploration has not been changed in this text and is as follows:

- I* Electric current flowing between two electrodes *C₁* and *C₂*.
- E* Drop in potential between two intermediate electrodes *P₁* and *P₂*.
- A* Electrode spacing.
- p* Specific resistivity in ohm-centimeters.
- L* Distance from point of percussion to detector.

- H* Depth of rock.
T Time required for wave to travel from point of percussion to detector.
V_e Velocity of wave through earth.
V_r Velocity of wave through rock.

Chapter IV

- H colloid Colloid ionized with hydrogen.
 Ca clay Clay ionized with calcium; calcium clay.
 HOH Hydrogen hydroxide or water.
 O Oxygen.
 OH Hydroxyl.
 KCl Potassium chloride.
 K Potassium.
 Cl Chlorine.
 ROH Any metallic hydroxide.
 pH Hydrogen-ion concentration.
 TH ' Total ionized hydrogen.

Chapter V

- d* Diameter of capillary tube.
ST Surface tension.
 α Angle of contact.
h Height of capillary rise.
D₁ Density of liquid.
g Gravitational constant.
SF Shrinkage force.
LL Liquid limit.
PL Plastic limit.
PI Plasticity index.
SL Shrinkage limit.
w Moisture content.
V Volume of wet soil.
V₀ Volume of dry soil.
W Weight of wet soil.
W₀ Weight of solids or dry soil.
SR Shrinkage ratio.
G 'Absolute' specific gravity of solids.
VC Volume change.
CME Centrifuge moisture equivalent.
FME Field moisture equivalent.
VC_f Volume change from *FME*.
LS Linear shrinkage.
F Flow index.
H Constant = intercept.
w Moisture content at *M* shocks.
TI = PI/F Toughness index.

Chapter VI

- HG* Hydraulic gradient.
h Loss of head.
d Distance in soil, thickness of soil sample, in which *h* is lost.
k_p Coefficient of percolation.
k Coefficient of permeability.
e Voids ratio.
v Velocity of flow.
a Area of standpipe.
A Area of soil face.
Q Discharge of water.
E_t Elevation of overflow from soil column.
t Time in seconds.
m Width.
L Length of reservoir.
E Percentage of voids.
T Temperature.
es Effective size.

Chapter VII

- B.t.u. British thermal unit.
Q Discharge of heat.
k Coefficient of conductivity.
A Area face of wall.
d Thickness of wall.
T₁ Temperatures.
T₂ Temperatures.
t Time in hours.
 C. Centigrade.
 F. Fahrenheit.
T_m Mean temperature.
D Depth below surface.
SH Specific heat.

Chapter VIII

- N* Number in a series.
P Load.
p Pressure.
r Radius, horizontal to a point in the ground from a load *P* on the surface.
z Vertical distance to a point in ground from location of load on surface.
p_s Pressure at point in ground located *r* and *z* distances from location of *P* on surface.
K Constant indicating effect of *r* and *z* on *p_s*.

Chapter IX

- U* Deflection of spring.
Q Total settlement of soil layer under load.

- y* Constant of spring deflection.
a Coefficient of compressibility.
*d*₀ Thickness of sample when $e = 0$.
*k*₀ Coefficient of permeability at a hydraulic gradient $\frac{h}{d_0}$ instead of $\frac{h}{d} = 1$.
c Coefficient of consolidation.
q Percentage of consolidation at any time t on the time-consolidation curve.
J Exponent in soil consolidation computations.
C Interpolation factor representing relation of c , t , and d_0 for soil sample in laboratory test, strata sandwiched between two permeable layers in field and for uniform or rectangular distribution of pressure in strata with permeable layer only at top.
D Thickness of soil stratum.
u Ratio pressure at top to that at bottom with trapezoidal distribution in stratum free to drain only from upper face.
*C*₁ Interpolation factor when $u < 1$.
*C*₂ Interpolation factor when $u > 1$.

Chapter X

- p* Pressure.
c Cohesion.
 ϕ Angle of internal friction.
 tan ϕ Coefficient of internal friction.
S Shear strength.
s Unit shear strength.
N Normal pressure.
 θ Angle of slope.
W Weight.
f Coefficient of friction.
H Critical height.
P' Perimeter of bearing area.
p' Resistance of soil to compression.
x Ratio of perimeter to area.

Chapter XVII

- H_w* Heat of wetting.
SH Specific heat.

APPENDIX III

GLOSSARY OF GEOLOGICAL TERMS

The following glossary of terms is from a report entitled "A Glossary of Geological Terms for the Highway Engineer" by D. G. Runner, published in *Roads and Streets*, March, 1936. The numbers in parentheses after the terms refer to the source or to literature where discussion of the subject matter may be found (page 406).

Aggregate.—The inert material, such as sand, gravel, shell, slag, or broken stone or combinations thereof, with which the cementing material is mixed to form a mortar or concrete (1).

Alluvium.—Deposits of mud and silt commonly found on the flat lands along the lower courses of streams.

Arenaceous.—From the Latin word "arena" meaning sand. Like, or pertaining to, sand. An example is arenaceous limestone, or a sandy limestone (2).

Argillaceous.—Containing or consisting of clay. An example is argillaceous limestone, meaning one containing a high percentage of clay (2).

Ashes (Volcanic).—Fragments of lava and pieces of rock driven upward by violent expansion and expulsion. The material usually hardens in its passage and falls in solid form.

According to size, those particles the size of peas are volcanic ashes (3).

Bank Gravel.—Gravel found in natural deposits, usually more or less intermixed with fine material, such as sand or clay or combinations thereof; gravelly clay, gravelly sand, clayey gravel, and sandy gravel indicate the varying proportions of the materials in the mixture (1).

Basalt.—Any volcanic rock of microcrystalline or amorphous texture, dark color, and high true specific gravity, indicating porous or volcanic basalt which by reason of its structure has a low apparent specific gravity (1).

Bed (of Rock).—A single layer or stratum of rock, which may be either an inch or 100 ft. or more in thickness.

Bedding Plane (of Rock).—Refers to the plane of junction between different beds or layers of rock (4).

Bentonite.—The plastic residue from the weathering of ash (volcanic); it swells enormously in water and forms a milky suspension (5).

Boulders.—Detrital material greater than 200 mm., or about 8 in., in diameter (4).

Caliche.—A material found chiefly in the southwestern part of the United States. It is composed essentially of soft limestone with varying percentages of clay (6).

Chats.—The gangue material which is found intimately mixed with the lead-zinc ores of Missouri and Oklahoma. It is closely akin to chert and is the by-product of metal mining.

Chert.—A very fine-grained, dense rock consisting of opal or chalcedony, often with some quartz and sometimes with accessory calcite, iron oxide, organic matter, sponge spicules, or other impurities. It is usually associated with limestones, either as entire beds or as isolated included masses. It has a homogeneous texture and a white, gray, or black color (7).

Clay.—A fine, argillaceous material which is more or less plastic when wet. The most important constituent of clay is hydrous aluminum silicate gel, a colloidal material, which remains suspended indefinitely in water but may be thrown down as coagulated matter by salt solutions. When dry, clay is a fine, earthy material having a characteristic odor when moistened by the breath. It clings to the tongue and makes a smooth paste when rubbed to an incoherent mass between the fingers. When pure, clay is white, but it is commonly colored yellow, brown, or red by iron oxides or gray, blue, or black by organic substances. The grain size of clay is commonly considered as minus 0.005 mm. (8).

Cleavage.—Is that capacity of certain rocks to part along parallel planes. It may be original in beds or secondary in metamorphic rocks (9).

Coquina.—Consists essentially of marine shells which are held together by a little calcium carbonate cement so as to form a fairly firm rock. It is full of cavities but is strong enough for use in building operations. The classic example is the "coquina" of the east coast of Florida.

Crusher Run.—Refers to the product of the crushing plant without being rescreened or separated into various sizes.

Diatomaceous Earth.—Is composed essentially of the siliceous skeletons of diatoms—extremely minute unicelled organisms. It is composed mainly of silica, white or light gray in color, and is extremely porous.

Dip (of Rock).—Is the angle of inclination of the plane of stratification with the horizontal plane.

Dolomite.—A magnesian limestone, composed essentially of the mineral dolomite—a double carbonate of calcium and magnesium. It is theoretically composed of 54 per cent calcium carbonate and 46 per cent magnesium carbonate (1).

Faults.—May be considered as an abrupt break in the continuity of the beds or strata with the elevation or depression of beds on one side of the plane of the fault (10).

Field Stone.—Loose rock commonly of assorted sizes and character found upon the surface of the ground. Such material is usually associated with near-by outcrops of rock.

Flint.—A sedimentary rock, dark gray to black in color, hard, dense, composed essentially of minutely crystalline silica with little chemically combined water. Flint is commonly found in chalk beds.

Gilsonite.—A hard, brittle, native asphalt occurring in various localities in rock crevices or veins from which it is mined like coal (11).

Gneiss.—A medium- or coarse-grained crystalline rock possessing some form of parallel structure due either to the uniform orientation of certain tabular or prismatic minerals or to the presence of wavy, discontinuous surfaces indicating a lenticular structure or of bands of varying mineralogical composition which retain their continuity and parallelism throughout a

considerable mass of rock. Some examples are banded gneiss, granite gneiss, etc. (12).

Grain (of Rock).—Refers to the direction of splitting at right angles to the "rift."

Granites.—Are crystalline, even-grained rocks consisting essentially of feldspar and quartz with smaller amounts of mica and other ferromagnesian minerals (13).

Gravel.—The granular, pebbly material (usually coarser than $\frac{1}{4}$ in. in diameter) resulting from the natural disintegration of rock. The rounded character of some gravel particles has been imparted by stream action.

Grit.—A coarse sand formed mostly of angular quartz grains (14).

Gumbo.—The natural soil found in certain parts of the Middle West, notably Minnesota, the Dakotas, Missouri, and Iowa, which consists of finely divided clays of varying capillarity (15).

Igneous Rocks.—Are those which have been formed or crystallized from molten magmas, the source of which has been the earth's interior. Examples of this type are granites, syenites, gabbros, etc. (13).

Indurated.—Refers to rocks that have been hardened by heat. Occasionally the term is applied also to sediments hardened by impregnating solutions (2).

Joints.—Traverse the rocks in different directions and at various angles, and in most areas there are at least two systems. The spacing of joints may vary greatly, sometimes being measured in inches, at other times in yards. Joints are a matter of practical importance, as the system and extent of this feature governs the size of the stone that can be removed from the quarry (3).

Kaolin.—A white, nonplastic material, consisting in the raw state of the mineral kaolinite (pure hydrated silicate of alumina) mingled with fragments of quartz, feldspar, mica, the residual minerals of granite (4).

Laminations.—Refer to the banding in rocks caused by variations in color of different minerals. Quite often these laminations result in the rock's breaking easily along the laminated portion. This feature is exemplified in the gneisses.

Lime Rock.—A natural material found chiefly in Florida and Georgia. It is composed essentially of calcium carbonate with varying percentages of silica. Lime rock hardens upon exposure to the elements, and some varieties provide excellent road metal (16).

Limestone.—Any natural rock of sedimentary origin composed principally of calcium carbonate or of calcium and magnesium carbonates in either its original chemical or fragmental or recrystallized form (1).

Loess.—Is wind-blown silt or silty clay having little or no stratification. Some of its peculiarities are the light color, fine state of subdivision, the sharpness and angularity of its particles, its porosity and coherence.

Marble.—Any crystalline or microcrystalline natural rock deposit of sedimentary origin composed principally of calcium carbonate or calcium and magnesium carbonates. This classification includes also those serpentines that are of igneous origin as well as those of sedimentary origin (1).

Marl.—An earthy mixture of minerals consisting of quartz, clay, calcite, and sometimes glauconitic sands. It is quite often found in swamps and in lakes.

Metamorphic Rocks.—Those rocks which have been changed by temperature, pressure, and chemical fluids into new forms, being more stable under the new conditions. Some examples are gneisses, schists, marbles, etc. (12).

Oolitic Limestone.—Those limestones consisting largely of minute spherical or ellipsoidal grains of calcium carbonate which resemble fish roe (4).

Outcrop (of Rock).—Those places where the underlying bedrock comes to the surface of the ground and is exposed to view.

Overburden.—Refers to the soil mantle, waste material, or other similar matter found directly above the deposit of rock or sand gravel. The amount of this overburden has a direct economic effect upon the amount of material capable of being produced.

Pebbles.—Rock fragments of small or moderate size which have been more or less rounded by erosional processes (17).

Quartzite.—A dense sandstone rock which has been thoroughly cemented by silica or which has been indurated or hardened by a recrystallization process.

Quartz.—Is composed entirely of silica, SiO_2 ; is colorless when pure, but often tinted yellow, red, blue, violet. It has a specific gravity of 2.65 and is the most common mineral in sand.

Red Dog.—Is the residue from burned coal dumps. The dumps are composed of waste products incidental to coal mining, which are not true coal but contain a certain percentage of carbon. Under pressure in the waste dumps, they frequently ignite from spontaneous combustion, and the residue is a red-colored ash containing a high percentage of silica with certain mineral salts distributed throughout the mass, which when wetted will form a firm and hard surface (15).

Rift (of Rock).—Quarrymen usually refer to the direction of easiest splitting as the "rift."

Rock Asphalt.—A porous rock which has become naturally more or less impregnated with asphalt or maltha. Some examples are Kentucky rock asphalt and the Uvalde rock asphalt of Texas (11).

Rubble.—Rough stones of irregular shapes and sizes, broken from larger masses either naturally or artificially, as by geological action, in quarrying, or in stone cutting or blasting (1).

Sand.—The fine, granular material (usually less than $\frac{1}{4}$ in. in diameter) resulting from the natural disintegration of rock or from the crushing of friable sandstone rocks (1).

Sand Clay (Road Surface).—A surface composed of a mixture of sand and clay where the two materials have been blended, so that their opposite qualities tend to maintain a condition of stability under varying moisture content. Some deposits are found in nature, but the bulk of such surfaces are prepared artificially (15).

Sandstone.—A typical sandstone is composed essentially of rounded grains of quartz, with or without interstitial cementing material, with the larger grains tending to be more perfectly rounded than the smaller ones. The

fracture in sandstones takes place usually in the cement, leaving the grains outstanding, thus giving the rock the appearance and feeling of loaf sugar.

Screenings.—Broken rock, including the dust, of a size that will pass through $\frac{1}{2}$ - to a $\frac{3}{4}$ -in. screen, depending upon the character of the stone (18).

Scoria.—Is applied to lava in which the gas cavities are numerous and irregular in shape. The escape of the gas distends the molten material which produces the cavities (4).

Schists.—Differ from gneisses in being of finer grain and in possessing a well-marked tendency to split into thin layers, except when puckered or folded by movement subsequent to the development of schistosity. Some examples of schists are mica schist, sericite schist, etc. (12).

Sedimentary Rocks.—Are formed by the decay of existing strata with subsequent deposition of the solid material that has been carried in suspension by agencies of transportation to some point where they have been redeposited (19).

Serpentine.—Refers to soft, greenish-colored rocks composed essentially of the mineral serpentine, or hydrous silicate of magnesia (10).

Shale.—A material composed essentially of silica and alumina, which has, in addition, a more or less thinly laminated structure. This structure has been imparted by the natural stratification of extremely fine sediments together with more or less pressure (20).

Shell.—The term is applied to oyster- or clamshells which have been dredged from the seas and used for a road-surfacing material.

Silt.—Consists of the finer particles of rock substance and ranges in size from 0.05 to 0.005 mm.

Slates.—Are rocks, normally clayey in composition, in which pressure has produced a very perfect cleavage, so that a block of slate can be readily split into thin, smooth, tough plates (10).

Soapstone.—Are rocks which usually consist of the mineral talc or some closely related mineral species. Talc is a hydrous magnesian silicate theoretically composed of 63 per cent silica, 32 per cent magnesia, and 5 per cent of combined water (10).

Soil.—A mixture of fine, earthy material with more or less organic matter resulting from the growth and decomposition of vegetation or animal matter (1).

Spall.—A small piece or fragment broken from a larger piece of rock.

Stone.—Any natural deposit or formation of igneous, sedimentary, and/or metamorphic origin, either in its original or altered form (1).

Stone Sand.—Refers to the product (usually less than $\frac{1}{4}$ in. in diameter) produced by the crushing of rock. This material is usually highly processed and should not be confused with screenings.

Strike (of Rock).—Is the direction of the line of intersection of the plane of stratification with the horizontal plane (3).

Subgrade.—The upper surface of the native foundation on which is placed the road metal or artificial foundation, in case the latter is provided (18).

Subsoil.—The bed of earth immediately beneath the surface soil (18).

Stratification (of Rock).—Is indicated by differences in composition, texture, hardness, cohesion, and/or color of the rock disposed in parallel bands (4).

Tailings.—Stones which after going through the crusher do not pass through the largest openings of the screen (1).

Talus.—A term used for the accumulation of fine, coarse, or mixed fragments and particles, fallen at or near the base of cliffs (21).

Texture (of Rock).—Is applied to the microscopic features, including the forms of crystals and the manner in which they are united (2).

Topsoil (Road Surface).—A variety of surfacing used principally in the southeastern states, being the stripping of certain topsoils which contain a natural sand-clay mixture. When placed on a road surface, wetted, and puddled under traffic it develops considerable stability (15).

Trap.—Includes the dark-colored, fine-grained, and dense igneous rocks composed essentially of the ferromagnesian minerals, basic feldspars, and little or no quartz. The ordinary commercial variety of trap is basalt, diabase, or gabbro.

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APPENDIX IV

TERMS IDENTIFYING SOILS IN THE PROFILE

The following terms are among those used in describing the various layers of the soil profile. They definitely identify the field characteristics of the soil material as found in its natural state. In the compilation of these terms the report of the committee on terminology of the American Soil Survey Association was largely used.

The terms describing structure, consistency, compactness, cementation, and chemical composition have not yet been standardized, nor have they been adopted by the division of soil survey, Bureau of Chemistry and Soils. They may be considered tentative and approximate. More accurate definitions are not yet available.

Texture

Texture is a term indicating the size of the individual soil grains or particles and the proportions of material of each size present in any given case.

As the soil is usually made up of particles of widely varying size, the textural terms express the average effect or the combined effect of all these grain sizes. They may indicate the predominance (in quantity or in textural effect) of a certain group of grains.

Texture is determined by mechanical analysis, a laboratory process of separating the soil into groups of grain sizes. The system of mechanical analysis used by the Bureau of Chemistry and Soils separates the soil material into seven grain sizes, or "separates," having the following sizes and names:

	Millimeters
Fine gravel.....	2 to 1
Coarse sand.....	1 to 0.5
Sand.....	0.5 to 0.25
Fine sand.....	0.25 to 0.10
Very fine sand.....	0.10 to 0.05
Silt.....	0.05 to 0.005
Clay.....	Below 0.005

In the following paragraphs are given the proportions of certain of the grain sizes found in the major soil textures:

Sands contain less than 20 per cent of silt and clay. (Include coarse, fine, and very fine sands.)

Sandy loams contain from 20 to 50 per cent of silt and clay but do not have over 15 per cent of clay. (Include coarse, fine, and very fine sandy loams.)

Loams have more than 50 per cent of silt and clay combined but have less than 50 per cent of silt and less than 20 per cent of clay.

Silt loams have more than 50 per cent of silt and less than 20 per cent of clay.

Clay loams have more than 50 per cent of silt and clay combined but less than 50 per cent of silt and between 20 and 30 per cent of clay. (Include sandy clay loams, clay loams, and silty clay loams.)

Clays have more than 50 per cent of silt and clay combined and more than 30 per cent of clay. (Include sandy clays and silty clays.)

In the field, texture is determined by the feel of the soil mass when rubbed between the fingers. The following statements give the obvious physical characteristics of the basic textural groups:

Sand.—Sand is loose and granular. The individual grains can readily be seen or felt. Squeezed in the hand when dry it will fall apart when the pressure is released. Squeezed when moist, it will form a cast but will crumble when touched.

Sandy Loam.—A sandy loam is a soil containing much sand but having enough silt and clay to make it somewhat coherent. The individual sand grains can readily be seen and felt. Squeezed when dry, it will form a cast which will readily fall apart; but if squeezed when moist, a cast can be formed that will bear careful handling without breaking.

Sands and sandy loams are classed as coarse, medium, fine, or very fine, depending on the proportion of the different-sized particles that are present.

Loam.—A loam is a soil having a relatively even mixture of the different grades of sand and of silt and clay. It is mellow, with a somewhat gritty feel yet fairly smooth and slightly plastic. Squeezed when dry, it will form a cast that will bear careful handling, while the cast formed by squeezing the moist soil can be handled freely without breaking.

Silt Loam.—A silt loam is a soil having a moderate amount of the fine grades of sand and only a small amount of clay, over half of the particles being of the size called "silt." When dry it may appear quite cloddy, but the lumps can be readily broken, and when pulverized it feels soft and floury. When wet the soil readily runs together and puddles. Either dry or moist it will form casts that can be freely handled without breaking. If squeezed between thumb and finger it will not "ribbon" but will give a broken appearance.

Clay Loam.—A clay loam is a fine-textured soil which breaks into clods or lumps that are hard when dry. When the moist soil is pinched between the thumb and finger, it will form a thin ribbon which will break readily, barely sustaining its own weight. The moist soil is plastic and will form a cast that will bear much handling. When kneaded in the hand it does not crumble readily but tends to work into a heavy, compact mass.

Clay.—A clay is a fine-textured soil that forms very hard lumps or clods when dry. When the moist soil is pinched out between the thumb and fingers, it will form a long, flexible ribbon.

Gravelly or Stony Soils.—All of the above classes of soils, if mixed with a considerable amount of gravel or stone, may be classed as gravelly sandy loams, gravelly clays, etc., as stony sandy loams, stony loams, etc., or as sandy clay loams, sandy clays, etc.

Floury.—Fine-textured soil consisting predominantly of silt (or flocculated clay with aggregates of silt size) which when dry is incoherent, smooth, and dustlike.

Gritty.—Containing a sufficient amount of angular grains of coarse sand or fine gravel, so that these dominate the "feel." Usually applied to medium-textured soils (loams) where the actual quantity of these coarse grains is rather small.

Heavy (Textured).—Applied to soils of fine texture in which clay predominates, with dense structure and firm to compact consistency. The term is also applied to soils containing a somewhat higher proportion of the finer separates than is typical of that textural class (as a "heavy sandy loam").

Light (Textured).—Applied to soils of coarse to medium texture with very low silt and clay content, incoherent, single-grained structure, and loose consistency. The term is also applied to soils containing somewhat higher proportions of the coarser separates than is typical of that textural class (as a "light loam").

Sharp.—Containing angular particles in sufficient amount to dominate the "feel." Abrasive.

Smooth.—Containing well-rounded coarser particles and a predominance of the finer separates. Not abrasive.

Color

In order to recognize the same soil in some other locality, the color should be clearly stated. This statement should give the range of color included, for classification purposes. By range of color is meant such terms as black or dark brown, brown to reddish brown, etc. The color may indicate the drainage conditions under which the soil was formed and the chemical composition of the soil.

In using such terms as grayish brown, brownish gray, etc., the adjective is recognized as a modifying term. The grayish-brown soil is a brown soil with a grayish cast sufficiently noticeable to require recognition; the brownish-gray soil is a gray soil with a brown cast.

Other color terms are as follows:

Mottled.—The presence of spots, streaks, or splotches of one or more colors in a soil mass of another predominant color. In mottled soils the colors are not mixed and blended, but each is more or less distinct in the general ground color. In color descriptions the ground color and the color of the included spots should be designated. Mottling is usually but not necessarily associated with poor drainage. The use of the term should not be confined to poorly drained soils but should be applied wherever the term fits.

Marbled.—The presence of two or more distinct colors in approximately equal amounts not blended but more or less mixed in occurrence in the soil mass. In a marbled soil there is no general or predominant color, as in the case of a mottled soil.

Spotted, Speckled, Streaked, Variegated.—Such terms can be used when their generally accepted meaning describes the color distributions that occur in the soils.

Structure

The term "soil structure" expresses the arrangement of the individual grains and aggregates that make up the soil mass. The structure may refer to the natural arrangement of the soil when in place and undisturbed (as structural profile) or to the soil at any degree of disturbance. The terms used indicate the character of the arrangement, the general shape and the size of the aggregates, and in some cases may indicate the consistency of those aggregates.

Adobe Structure.—This term describes a soil which on drying cracks and breaks into irregular but roughly cubical blocks. The cracks are usually wide and deep, and the blocks are from 20 to 50 cm. or more across. (Adobe soils are usually heavy textured and high in content of colloidal clay.)

Amorphous Structure.—A soil of fine texture having a massive or uniform arrangement of particles throughout the horizon. Structureless. Found only in soils of finest texture, where individual grains cannot be recognized.

Clod (or Cloddy) Structure.—Aggregates of irregular, angular shape, usually 4 cm. or more in diameter and of a hard consistency.

Fine Cloddy Structure.—When most of the clods are close to the minimum size.

Coarse Cloddy Structure.—When most of the clods are 10 cm. or more in diameter.

Columnar Structure.—A natural arrangement of the soil mass in more or less regular columns separated by vertical cleavage lines and usually broken by horizontal cracks into sections with longer vertical than horizontal axes, the tops of the columns being rounded.

Prismatic Columnar Structure.—Term used when the sections are very regular in size, straight sided, with the vertical axes much longer than the horizontal axes, and the tops of the columns flat.

Crumb Structure.—Porous aggregates of irregular shape, rarely over 2 cm. in diameter and of a medium to soft consistency.

Fine Crumb Structure.—Crumbs 5 mm. or less in diameter.

Coarse Crumb Structure.—Crumbs 2 cm. or more in diameter.

Crust (or Crusted) Structure.—This term is used where the upper or surface horizon coheres into plate or crust distinct from the horizon immediately below it.

Crust-mulch Structure.—An arrangement where a surface crust is underlain by a horizon of loose, incoherent particles of mealy, crumb, or granular structure.

Fluffy Structure.—A surface condition where the aggregates are loose, of light weight and fine texture, with no cohesion or evidence of arrangement; floury.

Dense Structure.—Having a minimum of pore space and an absence of any large pores or cracks. Approaching amorphous.

Granular Structure.—Aggregates varying in size to 2 cm. in diameter, of medium consistency, and more or less subangular or rounded in shape.

Fine Granular Structure.—Aggregates under 5 mm. in diameter.

Coarse Granular Structure.—Aggregates close to maximum size.

Honeycomb Structure.—A natural arrangement of the soil mass in more or less regular five- or six-sided sections separated by narrow or hairline cracks. Usually found as a surface structure or arrangement.

Hardpan.—A horizon of accumulation that has been thoroughly cemented to an indurated, rocklike layer that will not soften when wet. The term hardpan is not properly applied to hard clay layers that are not cemented or to those layers that may seem indurated when dry but which soften and lose their rocklike character when soaked in water. The true hardpan is cemented by materials that are not readily soluble and is a hard layer that definitely and permanently (in nature) limits downward movement of roots and water.

Clay Pan.—A horizon of accumulation or a stratum of stiff, compact, and relatively impervious clay. The clay pan is not cemented and if immersed in water can be worked to a soft mass. Its presence may interfere with water movement or root development the same as a true hardpan. It is more difficult to overcome, for, whereas a hardpan can be shattered by explosives, the clay pan, after breaking by any means, will run together and re-form as soon as thoroughly wetted. The distinction between hardpan and clay pan is an important one in the soil classification.

Laminated Structure.—An arrangement of the soil mass in very thin plates or layers, less than 1 mm. in thickness, lying horizontal or parallel to the soil surface. Usually medium to soft consistency.

Massive Structure.—A soil mass showing no evidence of any distinct arrangement of the soil particles. Structureless. May be found in soils of any structure.

Mealy Structure.—A crumblike structure in which the aggregates are of soft to very soft consistency and usually less than 5 mm. in diameter.

Nut Structure.—Compact aggregates, more or less rounded in shape, of hard to medium consistency, and from $\frac{1}{2}$ to 4 cm. in diameter.

Fine Nut Structure.—Aggregates below 1 cm. in diameter.

Coarse Nut Structure.—Aggregates over 3 cm. in diameter.

Plate (or Platy) Structure.—An arrangement of the soil mass in plates or layers 1 to 5 mm. or more in thickness, lying horizontal or parallel to the soil surface. Usually of medium to hard consistency.

Single-grained Structure.—An incoherent condition of the soil mass with no arrangement of the individual particles into aggregates. Structureless. Usually found in soils of coarse texture.

Structureless.—Without any discernible structure or arrangement of the soil particles into aggregates. This condition is better expressed by the terms single grained, massive, amorphous, etc.

Vesicular Structure.—A soil horizon or soil aggregate containing many small, rounded cavities smooth on the inside as though formed by gas bubbles.

Consistency

“Soil consistency” is a term expressing the degree of cohesion of the soil particles and the resistance offered to forces tending to deform or rupture the aggregates. Consistency and structure are closely related and frequently interdependent. The terms expressing consistency and structure are dis-

tinct, however, and need not be confused or used with double meaning. A study of published reports shows a general use of terms expressing both the consistency and the structure in nearly all soil descriptions.

Brittle.—A soil which when dry will break with a sharp, clean fracture. If struck a sharp blow, it will shatter into cleanly broken hard fragments.

Mellow.—Soil particles or aggregates are weakly adhered in a rather porous mass, readily yielding to forces causing rupture. A consistency softer than friable. Without tendency to pack.

Plastic.—Readily deformed without rupture. Pliable but cohesive. Can be readily molded. Putty-like. This term applies to those soils in which at certain stages of moisture the grains will readily slip over each other without the masses cracking or breaking apart.

Soft.—Yielding readily to any force causing rupture or deformation. Aggregates readily crushed between fingers.

Sticky.—Applied to soils showing a decided tendency when wet to adhere to other materials and foreign objects.

Firm.—Resistant to forces tending to produce rupture or deformation. Moderately hard. Aggregates can be broken between fingers.

Friable.—Aggregates readily ruptured and crushed with application of moderate force. Easily pulverized or reduced to crumb or granular structure.

Hard.—Resistant to forces tending to cause rupture or deformation. Difficult or impossible to crush aggregates with fingers only.

Tenacious.—Soils showing a decided resistance to rupture. Soil mass adheres firmly.

The terms "sticky" and "tenacious" are often used as synonyms, but in soil usage the former is taken to refer to adhesion, the latter to cohesion. Both terms may be applicable to a soil at the same time.

Stiff.—Resistant to rupture or deformation. A soil stratum or horizon that is firm and tenacious and tending toward imperviousness. Usually applied to condition of the soil in place and when moderately wet.

Tight.—A stratum or horizon that is compact, impervious, and tenacious and usually plastic.

Tough.—Resistant to rupture. Tenacious. A stratum or horizon that can be readily bored into with the auger but which requires much force in breaking loose and pulling out the core of soil.

Compactness

Compactness is the degree of resistance offered by a soil to the penetration of a pointed instrument.

Impervious.—Highly resistant to penetration by water and usually resistant to penetration by air and plant roots. Impenetrable. In field practice the term is applied to strata or horizons that are very slowly penetrated by water and that retard or restrict root penetration.

Indurated.—(See cementation.)

Loose.—Soil particles or small aggregates are independent of each other or cohere very weakly with a maximum of pore space and a minimum resistance to forces tending to cause rupture.

Cheesy.—Having a more or less elastic character, deforming considerably without rupture, yet broken without difficulty or the application of much force. (Characteristic of certain highly colloidal soils when thoroughly wet.)

Compact.—The soil packed together in a dense, firm mass but without any cementation. Noticeably resistant to forces tending to cause rupture or deformation. Coherent. Hard.

Relative degree of compactness may be expressed by terms as slightly compact, very compact, etc.

Cementation

Cementation.—A condition occurring when the soil grains or aggregates are caused to adhere firmly and are bound together by some material that acts as a cementing agent (as colloidal clay, iron or aluminum hydrates, lime carbonate, etc.).

The degree of cementation or the persistence of the cementation when the soil is wetted should be stated. Some terms indicate the permanence, as "indurated," "hardpan," etc.

Firmly Cemented.—Cementing material of considerable strength requiring considerable force to rupture the mass. Usually breaks, with clean though irregular fractures, into hard fragments.

Indurated.—Cemented into a very hard mass which will not soften or lose its firmness when wet and which requires much force to cause breakage. Rocklike.

Weakly Cemented.—Term applied when cementing material is not strong, and the aggregates can be readily broken into fragments with a more or less clean fracture.

Softly Cemented.—Term applied when cementing material is not strong or evenly diffused throughout the mass. Aggregates are readily crushed, but do not break with a clean fracture.

Chemical Composition

Peat Soil.—Composed predominantly of organic material, highly fibrous, with easily recognized plant remains.

Muck Soil.—Composed of thoroughly decomposed black organic material, with a considerable amount of mineral soil material, finely divided and with a few fibrous remains.

Leaf Mold.—The accumulation on the soil surface of more or less decomposed organic remains, usually the leaves of trees and remains of herbaceous plants. The A horizon.

Alkaline Soil.—A soil containing an excessive amount of the alkaline (in true chemical sense) salts.

Saline Soil.—A soil containing excessive amounts of the neutral or non-alkaline salts.

Calcareous Soil.—A soil containing sufficient calcium carbonate to effervesce when tested with weak (*N*/10) hydrochloric acid. Depending on the amounts present, these soils may be designated as slightly calcareous, strongly calcareous, etc.

Acid Soil.—A soil which is deficient in available bases, particularly calcium, and which gives an acid reaction when tested by standard methods. Field tests are made by the use of litmus, of Soiltex, and of other indicators. There is no full agreement on the most satisfactory test for acidity or as to the actual character of an acid soil. The intensity or degree of acidity may be expressed by qualifying words—"strongly," "moderately," etc.

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